Universidade de Aveiro Departamento de Engenharia Civil
2016

Dora Susana Gomes da Silveira

CARACTERIZAÇÃO CONSTRUTIVA E MECÂNICA DE PAREDES DE ALVENARIA DE ADOBE DE EDIFÍCIOS EXISTENTES

CONSTRUCTIVE AND MECHANICAL CHARACTERISATION OF ADOBE MASONRY WALLS OF EXISTING BUILDINGS

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Tese apresentada à Universidade de Aveiro para cumprimento dos requisitos necessários à obtenção do grau de Doutor em Engenharia Civil, realizada sob a orientação científica do Prof. Doutor Aníbal Guimarães da Costa, Professor Catedrático do Departamento de Engenharia Civil da Universidade de Aveiro, e coorientação do Prof. Doutor Humberto Salazar Amorim Varum, Professor Catedrático do Departamento de Engenharia Civil da Faculdade de Engenharia da Universidade do Porto.

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palavras-chave

conservação, reabilitação, construção em adobe, alvenaria de adobe, paredes de fachada, sistemas construtivos, anomalias de edifícios, propriedades mecânicas, comportamento cíclico, vulnerabilidade sísmica.

resumo

A construção em terra tem sido muito utilizada em todo o mundo, desde há cerca de 10000 anos atrás e até aos dias de hoje. Uma parte significativa do património mundial construído com terra, incluindo vários bens inscritos na Lista de Património Mundial da UNESCO, encontra-se, no entanto, em risco.

Em Portugal, a terra foi também um material de construção muito utilizado até meados do século XX. No distrito de Aveiro, em particular, a construção em adobe era muito comum. Atualmente, existe ainda um elevado número de construções em adobe nesta região, grande parte das quais se encontram em uso. Muitos dos edifícios existentes são de valor social, cultural e arquitetónico reconhecido. No entanto, apesar do seu valor, muitos destes edifícios estão em mau estado de conservação, apresentando anomalias estruturais e não estruturais variadas.

Os problemas observados nos edifícios existentes de adobe resultam, em grande parte, de falta de conhecimento sobre os materiais e sistemas de construção utilizados neste tipo de edificação. Há, em particular, falta de conhecimento sobre as propriedades e o comportamento das paredes de alvenaria de adobe, que são elementos estruturais principais que influenciam o comportamento global dos edifícios. Assim, o trabalho de investigação desenvolvido e discutido nesta tese tem como principal objetivo contribuir para este conhecimento, debruçando-se, em particular, sobre as construções em adobe do distrito de Aveiro.

Para este efeito, foi realizada uma inspeção visual e dimensional das paredes de fachada de vinte e um edifícios de adobe representativos. Com esta inspeção, foi possível analisar de forma detalhada as paredes de fachada – incluindo o seu sistema estrutural, revestimentos e materiais de alvenaria tradicionais – e avaliar as vulnerabilidades, anomalias comuns e estado de conservação destes elementos.

Uma série de ensaios experimentais foi também levada a cabo. Foram realizados ensaios de compressão simples sobre provetes cilíndricos e cúbicos de adobe, ensaios de flexão sobre blocos de adobe e ensaios de compressão diametral sobre provetes cilíndricos. Foram ainda realizados ensaios de compressão simples e compressão diagonal sobre dez painéis de alvenaria de adobe à escala real, construídos com adobes recolhidos de uma construção existente. Por fim, realizou-se o ensaio de uma parede à escala real em forma de ‘duplo T’, construída também com adobes de uma construção existente, sob a ação de uma carga horizontal cíclica, aplicada no plano da parede. Com os ensaios realizados, foi possível caracterizar a resistência, a rigidez, as relações de comportamento tensão-deformação e o padrão comum de dano dos elementos ensaiados, e avaliar as correlações entre diferentes propriedades mecânicas. Foi ainda desenvolvida uma comparação entre os valores de resistência obtidos e os limites indicados nas normas existentes para a construção em terra, bem como entre os resultados obtidos e aqueles determinados por outros autores para a alvenaria de adobe representativa da construção em adobe noutros países.

Os resultados apresentados e discutidos nesta tese contribuem para o enriquecimento de conhecimento que é considerado essencial para apoiar a conservação e reabilitação dos edifícios de adobe existentes, não só em Portugal, mas também noutras regiões do mundo.

keywords

conservation, rehabilitation, adobe construction, adobe masonry, facade walls, building systems, building defects, mechanical properties, cyclic behaviour, seismic vulnerability.

abstract

Earthen construction has been widely used throughout the world, since approximately 10000 years ago and until the present day. A significant part of the world earthen built heritage – including many properties inscribed on the UNESCO's World Heritage List –, however, is at risk.

In Portugal, earth was also a widely used construction material until the middle of the 20th century. In Aveiro district, in particular, adobe construction was very common. Currently, there are still a great number of adobe constructions in this region, a large part of which are in use. Many of the existing buildings are of social, cultural, and architectural value. Despite their value, however, many of these buildings are in a poor state of conservation, suffering from various structural and non-structural defects.

The problems observed in existing adobe buildings result in large part from a lack of knowledge regarding the materials and building systems used in this type of construction. There is, in particular, a lack of knowledge about the properties and behaviour of adobe masonry walls, which are key structural elements that influence the overall behaviour of buildings. The main aim of the research developed and discussed in this thesis is thus to contribute to this knowledge, focusing, in particular, on the adobe buildings of Aveiro district.

For this purpose, a visual and dimensional inspection of the facade walls of twenty-one representative adobe buildings was conducted. With this inspection, it was possible to carry out a detailed analysis of the facade walls – including their structural system, coatings, and traditional masonry materials – and to assess the vulnerabilities, common defects, and state of conservation of these elements.

A series of experimental tests were also carried out. Simple compression tests were performed on cylindrical and cubic adobe specimens, flexural tests on adobe bricks, and splitting tests on cylindrical specimens. Simple compression and diagonal compression tests were also conducted on ten full-scale adobe masonry wall panels, built with adobes taken from an existing construction. Finally, an in-plane horizontal cyclic test was performed on a full-scale double-T shaped adobe wall, also built with adobes from an existing construction. With the tests carried out, it was possible to characterise the strength, stiffness, stress-strain relationships, and common damage pattern of the test specimens, and to assess correlations between different mechanical properties. It was also possible to develop a comparison between the strength values obtained and the limits indicated in existing standards for earthen construction, and between the results obtained and those determined by other authors for test specimens representative of adobe construction in other countries.

The results presented and discussed in this thesis contribute to the enrichment of knowledge that is considered essential to support the conservation and rehabilitation of existing adobe buildings, not only in Portugal, but also in other regions of the world.

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Chapter 1

Introduction

1.1. Earthen construction: introduction

1.1.1. Earthen construction in the world

Earth is one of the oldest and most widespread building materials, having been used worldwide since ancient times and until the present day. A raw mixture of clay, silt, sand, and, sometimes, larger aggregates, is used, and other materials can be added to the mixture to improve its characteristics, such as fibres (e.g. straw, animal or human hair, and sisal fibre) and stabilisers (e.g. cement, lime, and bitumen) (Minke 2006). There are numerous earth building methods used throughout the world. Houben and Guillaud (1994) identify seven very commonly used methods: adobe, rammed earth, straw-clay, wattle and daub, direct shaping, compressed earth blocks, and cob. Also according to Houben and Guillaud (1994), among these, adobe, rammed earth, and compressed blocks are the most commonly used. These three building techniques have been subject to scientific studies and important technological developments since the last decades of the 20th century.

The use of earth as a construction material dates back approximately 10000 years, when the first homes and cities were built (Houben and Guillaud 1994). Adobe houses, built between 8000 and 6000 BC, were found in the former Russian Turkestan (Pumpelly 1908) and, in the Old Testament, many centuries before Christ, references to the fabrication of adobe were made. Earth was used, throughout the time, in almost all types of construction, from humble houses to palaces, from granaries to religious buildings, and even in military constructions (Gandreau and Delboy 2012). There are numerous examples of ancient earthen constructions that survived until the present day. For example, a large part of the Great Wall of China – which was built, approximately, between the 3rd century BC and the 17th century AD – is made with rammed earth (Gandreau and Delboy 2012). Another example is the citadel of Bam, in Iran, developed mainly between the 3rd and 18th centuries AD, which is made entirely with earth (Licciardi 2009). This citadel, however, suffered severe damage during the earthquake of December 2003.

According to Houben and Guillaud (1994), at the end of the 20th century, about 30% of the world population lived in earthen buildings and, in developing countries, this percentage rose to approximately 50%, including the majority of rural population and at least 20% of urban and suburban population. Presently, these figures should not be very different, with earth being used mainly in two ways:

i) On the one hand, the traditional earth building systems are still used in many developing countries – for example, in Africa, Latin America, and in some parts of Asia –, generally by the poorest segments of the population, due to the low cost and local availability of the material as well as the simplicity of earth building techniques;
ii) On the other hand, in some developed countries – such as the United States of America, some European countries, Australia, and New Zealand –, there is an increasing interest in earthen construction as a sustainable building alternative; in these countries, a significant number of earthen buildings have been built in the last decades, using traditional techniques that are adapted to meet modern structural and comfort demands, with the support of growing scientific research (Minke 2006; Correia et al. 2011).

A map with the distribution of earthen construction throughout the world is presented in Figure 1.1a. Many countries – including those in which earthen construction was completely, or almost completely, abandoned with the emergence of new industrial processes and materials – possess a vast and invaluable earthen built heritage, a significant part of which is still in use. This is the case of many countries in Europe, in which earth was one of the most commonly used construction materials until the 20th century

(Sandoval 2010) (Figure 1.2). For example, in Spain, earthen construction can be found in almost all areas of the country, being particularly abundant in the central region (Delgado and Guerrero 2006), and, in Germany, several hundred thousand traditional earthen buildings are currently preserved (Guérin et al. 2011).

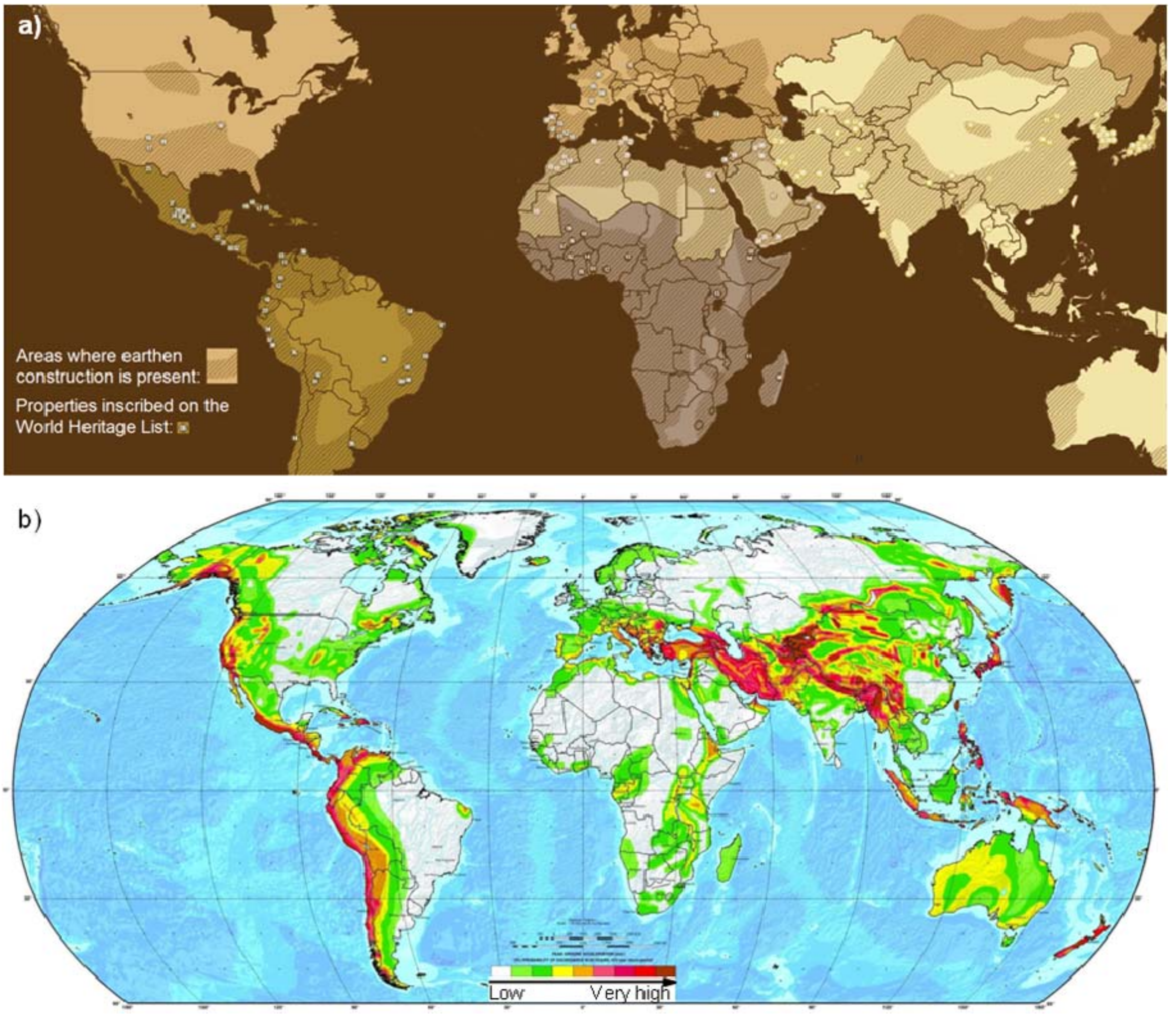

Figure 1.1: a) Distribution of earthen construction in the world and properties inscribed on the World Heritage List (adapted from Gandreau and Delboy (2012)); b) global seismic hazard map (Giardini et al. 1999).

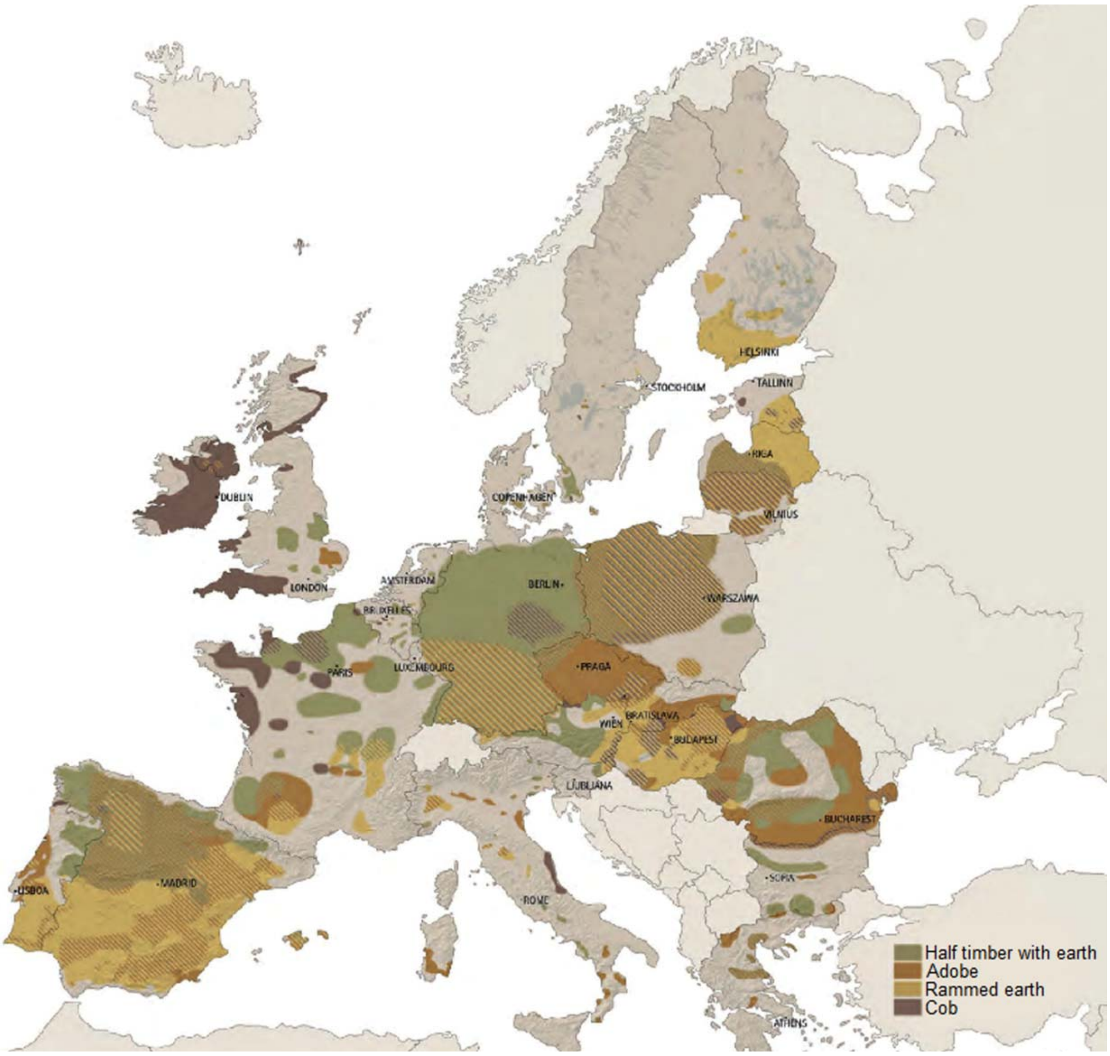

Figure 1.2: Distribution of earthen construction in Europe (Mileto et al. 2011).

The earthen built heritage that exists all over the world has great cultural value, meriting attention and protection by the international community. In 2012, in the framework of the World Heritage Programme on Earthen Architecture (WHEAP), a detailed inventory identified 150 properties partially or entirely built with earth inscribed in the World Heritage List (Figure 1.1a) (Gandreau and Delboy 2012). It was also observed that, in these properties, adobe is the most commonly used earthen technique. Considering the total number of cultural properties inscribed on the World Heritage List in 2015 (802, according to the United Nations Educational, Scientific and Cultural Organization (UNESCO 2016)), the properties partially or entirely built with earth correspond to at least 19% of that total number. Of the 150 identified properties, 15 (i.e. 10%) were, in 2015, included in the List of World Heritage in Danger, corresponding to 50% of the cultural properties included in this list (UNESCO 2016). Urgent action is thus needed in order to protect and preserve this valuable earthen built heritage.

1.1.2. Earthen construction in Portugal

In Portugal, earth was also a very common construction material until the middle of the 20th century. As a result, the country has a rich and varied earthen built heritage (Figure 1.3). The main traditional building techniques used were rammed earth, adobe, and *tabique* (wattle and daub).

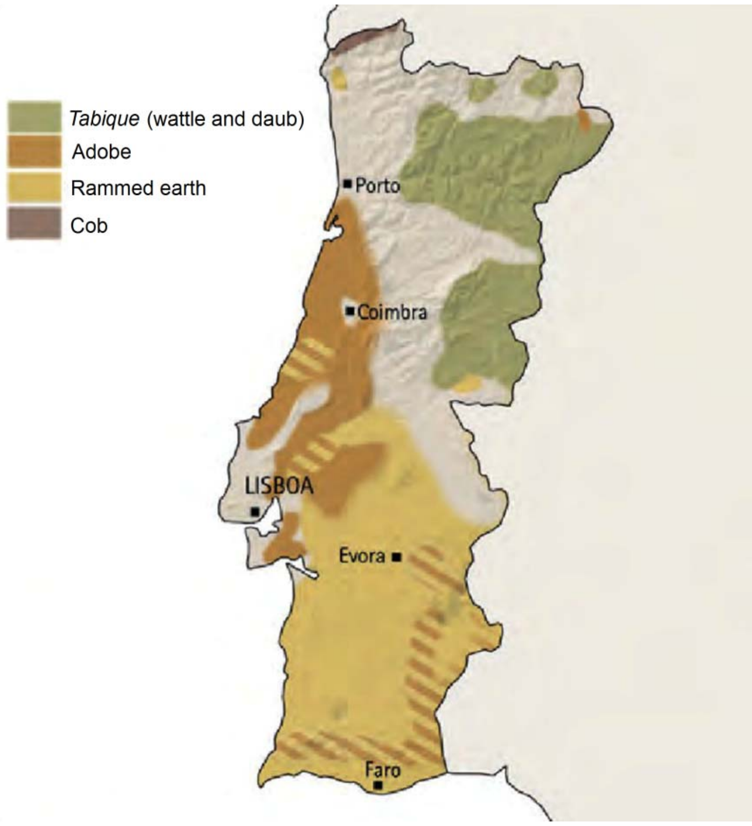

Figure 1.3: Distribution of earthen construction in Portugal (adapted from Correia and Merten (2011)).

Rammed earth was commonly used in the south of the country – in a large part of Alentejo and in some areas of Algarve and Ribatejo (Correia and Merten 2011). Adobe was used in Northern Estremadura, Ribatejo, and especially in Beira Litoral (Fernandes 2014; Tavares 1992). Adobe can also be found, even though less frequently, in the interior walls of houses in Alentejo and Algarve (Fernandes 2014). *Tabique* was used throughout the country in interior walls, being more common in the interior centre (Correia and Merten 2011). In the northern interior, it can also be found in the exterior walls of the upper floors of stone masonry buildings (Carvalho et al. 2008). Examples of construction with cob can also be found in Portugal but are rare – this building technique was recently identified in military fortresses in Alto Minho (Correia and Merten 2011).

Even though earthen construction has been almost completely abandoned in Portugal since the middle of the 20th century, in the last decades, there has been some new construction with earth, generally using the rammed earth technique, mostly in Algarve and the Alentejo coast (in the south of the country) (Correia and Merten 2011). This is in line with the trend observed in other countries where there has been a growing interest in this type of construction as a sustainable building alternative.

1.1.3. Advantages and vulnerabilities of earthen construction

The extensive use of earth as a construction material is associated with the many advantages of this material. Earth is low-cost (Morton et al. 2005), locally available, and reusable (Minke 2006). Earthen buildings generally provide good thermal comfort, due to the high thermal inertia of the earth material (Rodrigo et al. 2012; Desogus et al. 2014), and also good acoustic isolation (Morton et al. 2005). Earth also regulates air humidity, contributing to a balanced indoor climate (Morton et al. 2005; Minke 2006). Moreover, the preparation, transport, and use of earth on site require low energy consumption, producing little waste, very low carbon dioxide emissions, and almost no environmental pollution in general (Morton et al. 2005; Shukla et al. 2009).

Earthen construction, however, also has vulnerabilities that must be taken into account. This type of construction is particularly vulnerable to the action of weathering agents, especially to the action of water and wind (Qu et al. 2007; Bonazza et al. 2009), and thus needs adequate protection and regular maintenance measures (USDOI 1978). Moreover, earthen construction and, in particular, adobe construction, if not adequately designed and strengthened, may perform very poorly when subjected to seismic demands, suffering severe structural damage and often reaching collapse. This deficient behaviour is associated with the low tensile and shear strength and brittle behaviour of earthen structures (Yamín et al. 2003; ICG 2006). Earthen structures are also very heavy and, as a result, are submitted to large inertial forces during earthquakes (Blondet 2008). A global seismic hazard map is presented in Figure 1.1b, in parallel with the map displaying the distribution of earthen construction in the world (Figure 1.1a) – it is important to note that, even thought there are recent proposals of regional seismic hazard maps which are considered more accurate than the map presented (Giardini et al. 1999), for the purpose of this comparison the information provided by this map is sufficient. It can be observed that many regions where earthen construction is abundant are also areas with high seismic hazard, which suggests that a large part of the existing earthen constructions are at risk. The earthquakes that occurred in El Salvador, in January and February of 2001 (JSCE 2001), in Iran, in December of 2003 (EERI 2004; Mahdi 2005), in Peru, in August of 2007 (Blondet 2008), and in Chile, in February of 2010 (Elnashai et al. 2010; Contreras et al. 2011), are representative examples of the type of response that adobe constructions may present when subjected to seismic demands (Figure 1.4). In the 2001 El Salvador earthquakes, for example, more than one million people became homeless and the majority of damage occurred in adobe houses (JSCE 2001). The 2010 Chile earthquake and the subsequent tsunami caused damage to approximately 370000 buildings, of which about 37% were made of adobe (Elnashai et al. 2010). In the region of Maule, in particular, adobe construction was the most affected. In this region, in Curicó, for example, approximately 90% of adobe construction was destroyed (Elnashai et al. 2010).

Figure 1.4: Damage on adobe buildings caused by the earthquakes that occurred in: a) El Salvador, in 2001 (JSCE 2001); b) Iran, in 2003 (EERI 2004); c) Peru, in 2007 (Blondet 2008); d) Chile, in 2010 (Contreras et al. 2011).

Earthen construction is thus a valuable low-cost and sustainable building alternative; however, special care in the design, strengthening, and maintenance of this type of construction is fundamental. This should be taken into consideration not only in new constructions, but also in the conservation and rehabilitation of existing buildings.

1.2. Adobe construction in Aveiro district

1.2.1. Materials, diversity, and value

In a significant part of Aveiro district, in the Beira Litoral region, in Portugal, adobe construction was commonly adopted until the fifties and sixties of the 20th century (Oliveira and Galhano 1992; Tavares 1992). In these decades, this type of construction was progressively replaced by the use of ceramic brick masonry and reinforced concrete structures (Santiago 2005; Tavares et al. 2012).

The success of adobe construction in this region was a result, on the one hand, of the scarcity of stones for use in construction and, on the other hand, of the favourable characteristics of the existing available raw materials for use in the production of adobes. The main raw materials applied were coarse sand, argillaceous earth, and lime. The natural earth mixtures were corrected by the addition of clay or sand, and the addition of natural fibres to control cracking was also common. ‘Mud adobes’ were used at an early stage, and ‘lime adobes’ were used at a later stage, from the 19th century to the middle of the 20th century. From the middle of the 19th century, after a period of coexistence with ‘mud adobe’, ‘lime adobe’ started to prevail until it became the solution normally used (Santiago 2007). ‘Mud adobes’ were made with clayey soil, to which sometimes straw or other plant fibres were added (Oliveira and Galhano 1992; Santiago 2007). ‘Lime adobes’ were made with arenaceous soil (aggregate) (Santiago 2007) and air-lime (binder) in a percentage generally varying between 25% and 40% (Teixeira and Belém 1998). ‘Lime adobes’ could be made by dwelling owners for their own use (Oliveira and Galhano 1992), but the production of this type of adobe also assumed a semi-industrial level, with the existence of earth and lime suppliers and building contractors (Tavares 1992). There were many adobe production sites (*areeiros*) in the region, and those located in Esgueira were particularly important (Santiago 2005), extracting earth and producing adobes that were transported to Aveiro and surrounding areas and, from the Esgueira stream, to riverside areas from Ovar to Mira (Santiago 2007). Lime was produced in kilns located throughout the region, with particular abundance in the area of Cantanhede, in the neighbouring district of Coimbra (Tavares 1992; Mendes 2009).

Even though, at present, adobe is no longer used in this region, there are still a very significant number of adobe buildings, many of which are in use. In Aveiro city (considering the ‘Union of Parishes of Gloria and Vera Cruz’, i.e. the main area of the city), for example, according to a recent survey, it is estimated that about 40% of the existing buildings are made of adobe (Silveira et al. 2013). This percentage corresponds to 1330 adobe buildings, out of a total of 3388 existing buildings (considering the total number of buildings assessed in the 2011 census (INE 2011)). In the municipality of Murtosa, also according to a recent survey, it is estimated that approximately 25% of buildings are made of adobe (Silva 2012). This percentage corresponds to 1406 adobe buildings, out of a total of 5845 existing buildings (considering the total number of buildings provided by the 2011 census (INE 2011)). Adobe can be found in various types of construction, such as rural and urban buildings, churches, warehouses, walls for the delimitation of properties, water wells, and lime kilns (Figure 1.5). Many of the existing buildings are of social, cultural, and architectural value. The old Aveiro fire station building (Figure 1.5e), the old flour mill building converted by the University of Aveiro into a space for the promotion of scientific and technological culture (‘Fábrica Centro Ciência Viva’) (Figure 1.5f), and the National Republican Guard building (Figure 1.5g), among many others, are good examples of adobe buildings with important socio-cultural value. Among the buildings with architectural value, the numerous buildings influenced by the Art Nouveau style stand out. The ‘Major Pessoa House' (Figure 1.5a), recently converted into the Art Nouveau museum, is a key example.

Figure 1.5: Examples of adobe construction in Aveiro district: a) ‘Major Pessoa House’; b) urban houses; c) church building; d) warehouse; e) old fire station building; f) ‘Fábrica Centro Ciência Viva’; g) National Republican Guard building; h) land dividing wall; i) water well; j) rural house.

1.2.2. State of conservation

Despite their cultural and social value, a significant percentage of the existing adobe buildings in Aveiro district are in a poor state of conservation, displaying various structural and non-structural defects. In a recent survey focused on the main area of Aveiro city (i.e. on the ‘Union of Parishes of Gloria and Vera Cruz’), for example, it was concluded that approximately 30% of the existing adobe buildings are in a poor state of conservation (i.e. have defects that compromise their adequate performance or structural integrity) (Silveira et al. 2013). In Murtosa municipality, also according to a recent survey, this percentage rises to about 45% (Silva 2012). In some cases, maintenance and rehabilitation measures have been neglected in the last decades. In other cases, buildings that were subjected to recent interventions present defects caused by the inadequacy of the materials and techniques used (Maia 2009; Tavares et al. 2012). Thus, the problems observed in existing buildings result not only from negligence but also from a lack of knowledge concerning the characteristics and behaviour of the materials and building systems used in this type of construction.

It is also important to note that, since construction with adobe in this region was based on the accumulated experience, handed down from generation to generation, the building processes used generally did not include special attention to functional and comfort requirements nor a particular concern with seismic safety. As a result, many of the existing buildings are not adapted to meet current functional and comfort requirements and may not have sufficient seismic capacity.

1.3. Motivation and objectives

The world earthen built heritage – a large part of which, as seen previously, is built with adobe – is very significant and thus merits special attention by the international scientific community. Part of this built heritage, however, is at risk, as acknowledged by UNESCO (UNESCO 2016).

The existing adobe buildings in Aveiro district, in Portugal, specifically, have social, cultural, and architectural recognized value. Nevertheless, and as discussed in the previous subsection, many of these buildings are in a poor state of conservation due, in large part, to a lack of knowledge regarding the materials and building systems traditionally used. There is, in particular, a lack of knowledge concerning the properties and behaviour of adobe masonry walls. Adobe walls are key structural elements, which contribute to the global behaviour and performance of adobe buildings, and thus the good functioning of these elements is critical to the comfort and safety of building users. The existing lack of knowledge is especially true for lime stabilised adobe masonry. Literature specifically devoted to the study of traditional lime adobe masonry is scarce. Thus, constructions made with lime adobe, which are common not only in Portugal but also in other areas of the world (e.g. Dipasquale and Mecca (2011), Lopez et al. (2014)), require particular research attention.

The development of a thorough knowledge base to support the adequate conservation and rehabilitation of existing adobe buildings in Portugal and in other regions of the world is fundamental. These interventions are necessary to preserve this valuable built heritage and also to guarantee the comfort and safety of the persons who still use and live in adobe buildings.

The central objective of the research developed and discussed in this PhD thesis is thus to contribute to the knowledge about the properties and behaviour of the adobe masonry walls of existing adobe buildings. The research is focused, in particular, on the adobe buildings of Aveiro district, and its specific objectives are to contribute to:

i) Characterise the construction details of the facade walls of existing adobe buildings and evaluate their common defects and state of conservation;
ii) Characterise the mechanical properties and behaviour, in compression and tension, of adobes from existing constructions;
iii) Understand the influence of the experimental testing procedures used in the mechanical characterisation of adobes;
iv) Characterise the mechanical properties and behaviour, in simple and diagonal compression, of adobe masonry wall panels;
v) Understand the response of adobe structural walls when subjected to in-plane horizontal cyclic demands.

With the work developed it is thus intended to contribute to the enrichment of knowledge that can support: the development of further studies on the behaviour of adobe structures, such as the calibration of numerical models to simulate the performance of these structures; the development and testing of effective repair and retrofitting solutions; the creation of guidelines for the adequate conservation and rehabilitation of existing constructions; the development of technical standards focused on adobe construction, and the adaptation of existing standards to include this type of construction technique; and also the design of new adobe constructions. This knowledge is significant not only for the conservation and rehabilitation of adobe construction in Portugal, but also in other regions of the world.

1.4. Methodology

The methodology adopted to achieve the objectives defined can be summarized as follows:

i) Review of the existing literature on adobe construction, namely: research focused on the study of the building systems, defects, and vulnerabilities of existing adobe constructions; technical standards devoted to earthen and adobe construction; experimental studies of the mechanical properties and behaviour of adobe bricks and adobe masonry; and research focused on the experimental testing of the seismic behaviour of adobe structures;

ii) Visual and dimensional inspection of the facade walls of representative adobe buildings, with the use of inspection checklists adapted specifically for adobe construction, and analysis of the information gathered;

iii) Execution of experimental tests on adobe specimens, extracted from adobe bricks of existing constructions, and analysis of the results obtained, with the tests organised into two experimental campaigns:

   a) Simple compression and splitting tests conducted on cylindrical adobe specimens – to evaluate the compressive and tensile strength of the material, respectively;

   b) Simple compression tests performed on cylindrical and cubic adobe specimens, three point flexural tests on adobe bricks, and splitting tests on cylindrical specimens – to enrich the database about the mechanical properties and behaviour of adobe, and to assess the correlations between mechanical properties determined with different testing procedures;

iv) Execution of simple compression and diagonal compression tests on full-scale adobe wall panels, built with adobes collected from an existing construction and mortar with traditional composition, and analysis of the results obtained;

v) Execution of an in-plane horizontal cyclic test on a full-scale double-T shaped adobe wall, built with adobes taken from an existing construction and mortar with traditional composition, and evaluation of the respective results.

1.5. Organisation of the thesis

The present thesis is organized into seven chapters. The five main chapters correspond to research that was published, or submitted for publication, in peer reviewed international scientific journals. Small changes were made to the content of the articles to avoid repetitions and also to ensure consistency along the chapters of the thesis, in terms of, for example, structure and notation adopted. This first chapter provides an introduction to earthen construction, with special focus on adobe construction in Aveiro district, and presents the motivation, main objectives, and organisation of the research developed and addressed in the thesis. The second chapter describes the results of the visual and dimensional inspection of the facade walls of twenty-one representative adobe buildings. The third and fourth chapters present the experimental analysis of the mechanical properties and behaviour of adobe specimens and evaluate the correlations between results obtained with different testing procedures – since these studies were conducted in two phases and published in two different articles, it was decided to maintain that separation in the thesis. The fifth chapter analyses the results of simple compression and diagonal compression tests conducted on full-scale adobe wall panels. The sixth chapter addresses the results obtained in the testing of a full-scale double-T shaped adobe wall, submitted to in-plane horizontal cyclic loading of increasing amplitude. Finally, the seventh chapter summarizes the main conclusions of the research and suggests possible future developments. The content of each chapter is described in more detail in the next paragraphs.

The second chapter presents a brief description of the methodology, developed in three levels of increasing detail, created for the survey and characterization of the adobe constructions in Aveiro district. A map with the distribution of adobe construction in Aveiro district is proposed (‘Level 1’). A detailed description and analysis of the facade walls (including the structural system, coatings, and traditional masonry materials) of twenty-one representative adobe buildings, selected from three municipalities in Aveiro district, and an assessment of the common defects and state of conservation of these structural elements are presented (‘Level 3’).

The third chapter reports the results of the first experimental campaign for the mechanical characterisation of adobe specimens. In this campaign, simple compression and splitting tests were conducted on cylindrical adobe specimens extracted from adobe bricks taken from representative constructions in Aveiro district. The compressive and tensile strength values obtained from the tests are discussed. A comparison of the strength values obtained with the strength limits indicated in different technical standards for earthen construction is carried out. A comparison of the results with those obtained by other authors for adobes representative of adobe construction in different countries is also presented. The correlation between the tensile strength and compressive strength of the specimens is studied.

The fourth chapter presents the results of the second experimental campaign for the mechanical characterisation of adobe specimens. In this campaign, simple compression tests were performed on cylindrical and cubic adobe specimens, three point flexural tests on adobe bricks, and splitting tests on cylindrical specimens; as in the first experimental campaign, the test specimens were extracted from adobe bricks collected from representative houses in Aveiro district. The following parameters are evaluated: compressive strength; flexural and splitting tensile strength; strain at peak stress; modulus of elasticity; and Poisson's ratio. Three different theoretical stress-strain curves, calibrated with the results obtained, are proposed. Correlations between the results obtained with different testing procedures are determined. The correlation between modulus of elasticity and compressive strength is also studied.

The fifth chapter describes the results of simple compression and diagonal compression tests performed on ten full-scale adobe wall panels, constructed in the laboratory with adobes collected from a representative construction in Aveiro district and mortar similar to that traditionally used. The stress-strain relationships, compressive and shear strength, modulus of elasticity, modulus of rigidity, Poisson's ratio, and damage pattern of the adobe walls are studied. Two theoretical stress-strain curves are proposed as approximate representations of the curves determined for the adobe panels tested in simple compression. A comparison of the strength values obtained with the strength limits indicated in the Peruvian technical standard for adobe construction is carried out.

Comparisons of the results obtained with those determined by other authors for adobe wall panels representative of existing construction in other countries are also presented.

The sixth chapter addresses the results of an in-plane horizontal cyclic test conducted on a full-scale double-T shaped adobe wall, built with adobes taken from a representative construction in Aveiro district and mortar produced with composition similar to that traditionally used. The behaviour of the wall is assessed in terms of: shear stress versus horizontal drift and moment versus rotation relationships; maximum lateral strength; drift and rotation at peak stress; evolution of stiffness, lateral displacement, dissipated energy, and natural frequency; and damage pattern. A comparison of the results obtained with those determined by other authors in in-plane cyclic or monotonic tests performed on simple or double-T shaped adobe walls, representative of existing construction in different countries, is also presented.

Finally, the seventh chapter presents a summary of the main conclusions of the research developed. Some suggestions for future work are also provided.

1.6. References

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Chapter 2

Survey of existing adobe buildings

The work reported in this chapter is presented in: Silveira, D., Varum, H., Costa, A., and Neto, C. "Survey of the facade walls of existing adobe buildings." *Int. J. Archit. Herit.* (accepted for publication on 10 February 2016).

2.1. Introduction

Knowledge about the material and building systems, defects, and vulnerabilities of existing adobe buildings is critical to support their conservation and rehabilitation. A study of the facade walls of twenty-one representative adobe buildings, located in Aveiro district, in Portugal, was conducted with the aim of contributing to expand this knowledge. The results obtained in this study are presented in this chapter.

A brief review of previous research developed to contribute to this knowledge, focused on existing adobe construction worldwide and, in particular, in Portugal, and a further description of the motivation and summary of the present work are presented below.

2.1.1. Research on adobe construction worldwide

Adobe construction, if not effectively designed and strengthened, may perform very poorly when subjected to seismic demands (Blondet 2008). Thus, one of the main aims of existing scientific research on earthen and, in particular, adobe construction has been the study of its seismic behaviour and the development of effective seismic retrofitting solutions, mainly by conducting experimental laboratory work, with the first studies dating back to the early seventies (e.g. Tolles (2009), Blondet et al. (2011)).

Other types of research focused on existing earthen construction, however, have also been conducted. An effort to address the earthen built heritage in different regions of the world has been made by the international scientific community, in an attempt to document and preserve this valuable heritage and, in regions where this material is still used, to promote good building practices.

The careful study of the material and building systems, defects, and vulnerabilities of existing earthen constructions has been deemed essential for their adequate preservation and rehabilitation. Various studies of adobe construction, in particular, have been conducted, focusing on regions of the world where this type of construction is especially abundant, namely:

- Latin America (e.g. Rivera and Muñoz (2005), Lardinois and Cancino (2012), Rolón and Rotondaro (2012), Jorquera (2013));
- Asia (e.g. Fodde (2009), Mecca and Dipasquale (2012), Pozzi (2012));
- Mediterranean Europe (e.g. Bosia (2009), Oikonomou and Bougiatioti (2011), Gil (2014));
- Africa (e.g. Abufayed (2005), Baglioni et al. (2013)).

Some of the studies carried out are focused only on the characterisation of the material and building systems (e.g. Pozzi (2012), Baglioni et al. (2013)), while other studies also assess existing defects (e.g. Bosia (2009), Mecca and Dipasquale (2012)), seismic vulnerability (e.g. Lardinois and Cancino (2012), Jorquera (2013)), and hygrothermal behaviour (e.g. Oikonomou and Bougiatioti (2011)). These studies generally focus on the existing construction in large geographical areas such as a country (e.g. Jorquera (2013)), province (e.g. Gil (2014)), or city (e.g. Abufayed (2005)), frequently including the analysis of a sample of representative buildings. Some of the studies address only adobe construction (e.g. Abufayed (2005)), while others also address other types of earthen construction (e.g. Gil (2014)). In some of the building typologies, adobe is combined with other building techniques, such as rammed earth, stone masonry, or wooden structures (e.g. Oikonomou and Bougiatioti (2011), Baglioni et al. (2013)). In some cases, methodologies for the assessment and rehabilitation of existing buildings have been proposed (e.g. Rivera and Muñoz (2005), Bosia (2009)).

2.1.2. Research on adobe construction in Portugal

2.1.2.1. Overview

The existence of earthen construction in Portugal has been recorded in early studies conducted between the thirties and seventies of the 20th century, focused on the existing traditional construction (Barros 1947; Ribeiro 1992; Oliveira and Galhano 1992; AAP 1988). These studies provide a broad and general view of the Portuguese traditional construction until the sixties (Fernandes 2013) but only include brief descriptions of the materials and building systems used in the buildings studied.

Recently, the scientific community has begun to recognise the importance to address the existing earthen built heritage in Portugal, with the aim of creating knowledge that may support its conservation and rehabilitation. The research dedicated, in particular, to the study of adobe construction in Portugal can be categorised according to different areas of focus. Some of the main areas of focus of this research are as follows:

- Study of the history of the use of adobe as a building material (e.g. Santiago (2007), Bruno (2010), Carvalho (2013), Fernandes (2013));
- Study of the architecture of the different adobe building typologies (e.g. Fernandes and Mestre (2006), Santiago (2007), Maia (2009), Tavares (2009), Fernandes (2013));
- Study of the materials and processes traditionally used in the production of adobe bricks (e.g. Santiago (2007), Maia (2009), Fernandes (2013));
- Study of the composition and behaviour of adobe bricks and traditional renders and joint mortars (e.g. Coroado et al. (2010), Almeida (2012), Silveira et al. (2012), Velosa et al. (2012), Silveira et al. (2013a), Martins (2015));

- Study of the mechanical properties and behaviour of adobe walls and structures (e.g. Varum et al. (2007), Rufo (2010), Varum et al. (2011), Almeida (2012), Oliveira et al. (2012), Martins (2015), Silveira et al. (2015));
- Study of the hygrothermal properties and behaviour of adobe constructions (e.g. Meneses (2010), Parracho (2011), Cancela (2013));
- Study of the building systems and common structural and non-structural defects of existing adobe buildings (e.g. Santiago (2007), Ferreira (2008), Maia (2009), Tavares (2009), Fernandes (2013), Martins (2015)) – these studies will be presented in more detail in the following subsection (2.1.2.2);
- Development of maintenance and rehabilitation solutions and guidelines for existing adobe buildings (e.g. Silva (2012a), Tavares et al. (2014), Velosa and Varum (2014), Andrejkovičová et al. (2015));
- Development of structural retrofitting solutions for existing adobe buildings (e.g. Figueiredo et al. (2013)).

2.1.2.2. Study of the building systems and defects of adobe buildings

Regarding the study of the building systems, defects, and vulnerabilities of existing adobe buildings, there are a number of studies carried out with different objectives and focused on different geographical areas of the country. Some of the main studies are summarised below.

Aveiro municipality

The building systems and main structural defects of masonry buildings located in two distinct areas of Aveiro city – the great majority of which were made with adobe – were studied with the objective of assessing the seismic vulnerability of these urban areas (Ferreira 2008).

Another study of 120 adobe buildings located in Aveiro city was carried out. These buildings were subject to an expeditious visual inspection conducted mainly from the outside and the most important defects were recorded and analysed (Martins 2009; Martins 2015). This study also included the geometric and constructive characterisation of a sample of representative buildings located in different parts of Aveiro district (Martins 2015).

A survey of the existing adobe construction in the parish of Requeixo, in Aveiro municipality, was also conducted (Maia 2009). The research was focused on the study of the traditional production and construction processes and also on the characterisation of the architecture, building systems, and main defects of selected buildings, including a brief suggestion of adequate rehabilitation measures.

Ílhavo municipality

A study of the adobe buildings in Ílhavo city, built in a phase of transition of the language of architecture, in the beginning of the Modernist movement, was carried out (Tavares 2009; Tavares et al. 2012). The study was focused on the architecture, building systems, and main defects of the existing buildings and also on the influence of public and private agents in the transition process. It included brief guidelines for the rehabilitation of the buildings studied.

Other regions

A study of the Gandaresa adobe house – a traditional rural housing typology – was conducted (Santiago 2007). The study was focused on the existing constructions in the coastal territory between the Vouga river and the Boa Viagem mountain range. It addressed the historical and geographical context of the Gandaresa house, the types of soil and processes used in the production of adobes, and the architecture, building systems, and main defects of this type of construction.

Finally, a study of the adobe building culture in Portugal was carried out (Fernandes 2013). This work included a broad investigation of the adobe architecture, materials, and building methods in Portugal. It also included a reflection on the importance of the conservation of the existing adobe buildings and on the possibility of future adobe production and construction in Portugal.

2.1.3. Motivation and summary

Each region of the world has unique earth building materials and techniques that must be assessed independently. However, there are also common features and vulnerabilities among different building cultures that are relevant to the conservation of the earthen built heritage in general. The assessment of the existing built heritage is an ongoing process, and further attention by the international scientific community is necessary.

The research that has been conducted in Portugal, in particular, provides an important contribution to the knowledge regarding the material and building systems of the existing adobe buildings. However, further work developed in a more systematic and comprehensive way is necessary to better characterise and understand this type of construction. This knowledge is essential to support the creation of solutions and guidelines for the preservation and rehabilitation of this built heritage. It is relevant for the conservation of the existing adobe construction not only in Portugal but also in other regions of the world.

To contribute to the existing knowledge, a research group at the University of Aveiro, in collaboration with other institutions, has been carrying out a thorough survey of the existing adobe constructions in Aveiro district. The research is conducted using a methodology developed in three levels of increasing detail, described in the next subsection. This chapter addresses the analysis of part of the information gathered in ‘Level 3’, namely the information resulting from the visual and dimensional inspection of the facade walls of twenty-one adobe buildings. These buildings, located in Anadia, Murtosa, and Aveiro municipalities, are representative of the existing adobe construction in Aveiro district. A detailed description and analysis of the facade walls (including the structural system, exterior wall finishes, and traditional masonry materials) and an analysis of their common defects and state of conservation were carried out. Adobe facade walls are key structural elements, responsible for the overall behaviour and performance of adobe buildings. The present work aims to contribute with preliminary information that may support the effective rehabilitation and proper functioning of these key structural elements.

2.2. Methodology

The strategy adopted in the survey of the existing adobe constructions in Aveiro district is developed in three levels of increasing detail. In a first phase (‘Level 1’), the city councils of all the municipalities of Aveiro district are contacted to obtain information about the distribution of adobe construction throughout the district. This information is obtained using a brief questionnaire completed by technicians working in the management of the existing built heritage (two examples of completed questionnaires are presented in Appendix A). This first phase of the analysis is currently complete. With the information obtained from the city council technicians and from in situ observations made in each municipality, it was possible to create a map with a first proposal for the distribution of adobe construction, by municipality, in Aveiro district (Figure 2.1). The municipalities where adobe construction is more abundant are: Murtosa, Aveiro, Ílhavo, Vagos, and Oliveira do Bairro. In these municipalities, adobe was the construction material commonly used until the middle of the 20th century. The municipalities of Ovar, Estarreja, Albergaria-a-velha, Águeda, and Anadia are regions of transition of construction materials. In some parishes of these municipalities adobe construction is still very abundant, but in others it coexists with stone construction, and in others it is almost nonexistent or even nonexistent. In all the other municipalities of Aveiro district, stone was the construction material traditionally used, and adobe construction is almost or completely nonexistent.

In a second phase (‘Level 2’), a broad survey of the distribution, main characteristics, and global state of conservation of existing adobe buildings in selected parishes is conducted. The selection of these parishes is based on the information gathered in ‘Level 1’. The survey method is expeditious and consists in the visual inspection of the outside of each building, accompanied by a brief photographic and written record of the main features of the building, making use of survey forms specifically developed for this purpose (an example of survey form is presented in Appendix B). At present, the following surveys have been conducted:

i) Survey of the existing adobe buildings in the former parishes of Gloria and Vera Cruz (since 2013, ‘Union of Parishes of Gloria and Vera Cruz’), in Aveiro municipality, with 1330 adobe buildings identified (Silveira et al. 2013b);

ii) Survey of the existing adobe buildings in all the parishes of Murtosa municipality, with 1406 adobe buildings identified (Silva et al. 2010; Silva 2012b).

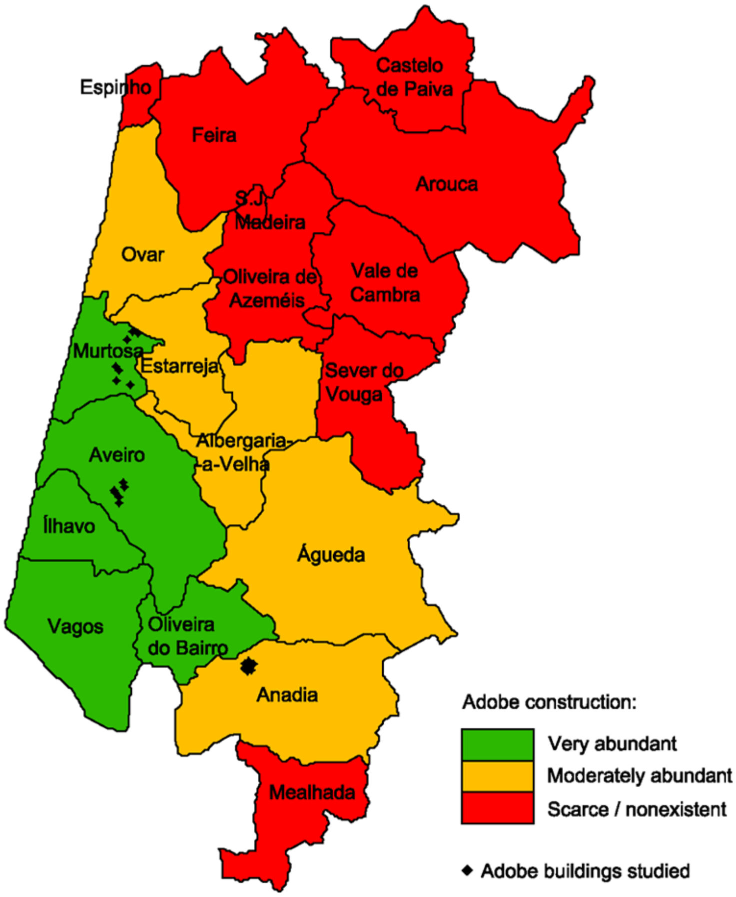


Figure 2.1: Distribution of adobe construction in Aveiro district and location of the buildings studied.

The knowledge gained in the first two analyses ('Level 1' and 'Level 2') allows the selection of buildings that are representative of the existing construction in Aveiro district for a more detailed study of materials, building systems, and structural and non-structural defects ('Level 3'). This study is carried out with the application of a set of inspection checklists, which were adapted specifically for adobe construction from existing inspection checklists developed by Vicente (2008) for the study of the buildings of downtown Coimbra. These checklists include the following information:

i) Identification of the building (i.e. main identifying characteristics of the building, such as GPS coordinates, setting, year of construction, number of stories, and function);

ii) Evaluation of the characteristics and defects of the different building systems – roof, facade walls, floors, interior walls, ceilings, foundations, and masonry materials (adobe and traditional mortars);
iii) Assessment of the surrounding land and vicinity of the building.

The analysis of each building is conducted by a team generally composed of two persons (with appropriate training), during approximately one to two working days. The analysis consists of a visual inspection combined with basic measurements (carried out using a laser distance measurer, tape measure, and level) and a detailed photographic record. The information obtained is recorded in the aforementioned checklists.

This chapter presents the analysis of part of the information gathered in 'Level 3'. It focuses on the results of the inspection of the facade walls (including foundations and traditional masonry materials) of twenty-one selected adobe buildings. Part of the information about the identification of the buildings is also presented. The inspection checklists that were used in this analysis are presented in Appendix C.

2.3. Identification of buildings

From 2007 to 2011, twenty-one adobe buildings in Aveiro district were subject to a thorough visual and dimensional inspection. The buildings are presented in Figure 2.2, with indication of the identification number assigned to each building. Table 2.1 displays the distribution of the buildings by municipality and parish. The type of setting (rural or urban) is also indicated. Figure 2.1 displays the location of the buildings within the district. The buildings were selected from three municipalities: Anadia (seven buildings); Murtosa (seven buildings); and Aveiro (seven buildings). The main reasons for the choice of these locations are as follows:

i) Adobe construction is very common in the selected parishes (as observed in 'Level 1' of the study), since it was the solution normally adopted until the middle of the 20th century;
ii) There is diversity in the setting and location of the selected areas: Anadia and Murtosa municipalities are predominantly rural, and the selected parishes of Aveiro

municipality are urban (corresponding to the main area of Aveiro city); Murtosa and Aveiro are coastal municipalities, located in the north of the district area where adobe construction is more common (Figure 2.1), and Anadia is an interior municipality, located at the southern boundary of that area.

Figure 2.2: Adobe buildings analysed in: a) Anadia municipality; b) Murtosa municipality; c) Aveiro municipality.

Table 2.1: Distribution of the buildings studied by municipality, parish, and type of setting.

Municipality	Parish	Setting	No. of buildings
Anadia	Sangalhos	Rural	7
		Total	7
Murtosa	Bunheiro	Rural	3
	Monte	Rural	1
	Murtosa	Rural	1
		Urban	2
		Total	7
Aveiro	(Former) Glória [a]	Urban	5
	(Former) Vera Cruz [a]	Urban	2
		Total	7

[a] The former parishes of Glória and Vera Cruz, officially united and named 'Union of Parishes of Gloria and Vera Cruz' in 2013, constitute the main area of Aveiro city.

Some of the main characteristics of the buildings studied are presented in Table 2.2. The majority of the buildings (57%) are detached. The second most common type of relative position is semi-detached or end-of-terrace (33%). In Anadia and Murtosa municipalities, the majority of the selected buildings are detached. This is consistent with the reality of these two municipalities, which, being mainly rural, have a greater percentage of detached buildings (e.g. approximately 60% in Murtosa municipality, according to Silva (2012b)). Since Aveiro city is an urban setting, terraced, semi-detached, and end-of-terrace buildings are more common (approximately 75%, according to Silveira et al. (2013b)). This was taken into account in the selection of the buildings in Aveiro city and, considering the existing buildings that were available for the execution of the inspections, the semi-detached and end-of-terrace positions prevail.

The function recorded for each building corresponds to its past function, in case it was vacant at the time of the inspection, or to its current function, in case it was in use. The great majority of the selected buildings are residences (76%), since this is the most common type of adobe building in Aveiro district – for example, approximately 90% of adobe buildings in Murtosa municipality and 70% in Aveiro city are residences, according to Silva (2012b) and Silveira et al. (2013b), respectively. The function of the adobe buildings in the selected parishes of Aveiro municipality is slightly more varied than in the rural areas of the district, and thus buildings with other uses were also chosen in these parishes. In this municipality, in two cases (‘*H*37’ and ‘*H*40’) residence was combined with commerce or services (located on the ground floor), and one building (‘*H*38’) accommodated a warehouse, commerce, and services. Buildings ‘*H*33’, in Murtosa municipality, and ‘*H*41’, in Aveiro municipality, were about to undergo rehabilitation and currently function as municipal archive and headquarters of the Federation of Fire-fighters of Aveiro district, respectively. The photo of building ‘*H*41’ presented in Figure 2.2 was taken after the rehabilitation intervention, since during the visit conducted the building was covered with scaffolding.

Table 2.2: Relative position, function, and number of stories of the buildings analysed.

		No. of buildings			
		Anadia	Murtosa	Aveiro	Total
Relative position	Detached	4	6	2	12 (57%)
	Terraced [a]	1	0	1	2 (10%)
	Semi-detached / end-of-terrace	2	1	4	7 (33%)
Function	Residence	7	6	3	16 (76%)
	Residence and commerce /services [b]	0	0	2	2 (10%)
	Warehouse and commerce/services	0	0	1	1 (5%)
	Originally: residence; presently: municipal archives	0	1	0	1 (5%)
	Originally: fire station; presently: headquarters of the Federation of Fire-fighters	0	0	1	1 (5%)
No. of stories	1	4	1	2	7 (33%)
	2 [c]	2	5	4	11 (52%)
	3 [c]	1	0	1	2 (10%)
	4	0	1	0	1 (5%)

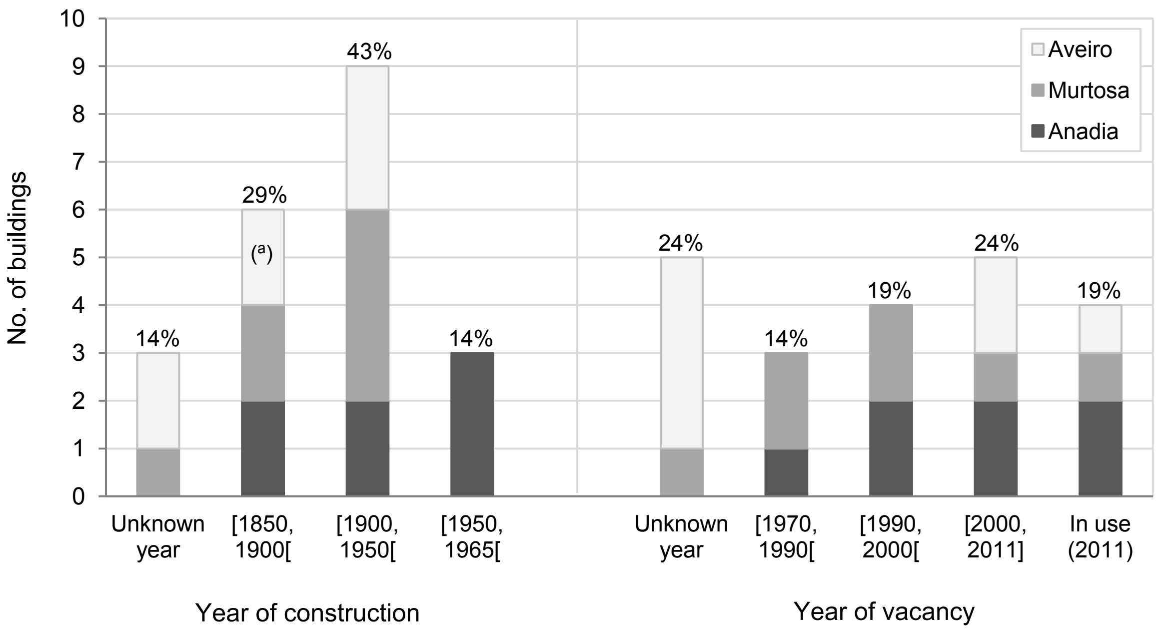
[a] ‘Terraced’ is used to mean a building that is part of a row of buildings joined together (with at least one building attached to each side).

[b] The commerce/services section is located in part of the ground floor of the building.

[c] Two buildings with two stories (‘*H*25’, in Anadia municipality, and ‘*H*39’, in Aveiro municipality) and one building with three stories (‘*H*27’, in Anadia municipality) have semi-basements, which are included in the total number of stories.

The great majority of the selected buildings (86%) have one or two stories, which is representative of the reality of Aveiro district – for example, approximately 98% of adobe buildings in Murtosa municipality and 80% in Aveiro city have one or two stories, according to Silva (2012b) and Silveira et al. (2013b), respectively. Fewer buildings with three and four stories were analysed, since adobe buildings with more than two stories are less common. In Aveiro city, adobe buildings with two stories predominate (Silveira et al. 2013b), but in Murtosa municipality adobe buildings with one storey are more common (Silva 2012b). The selection of more two-storey buildings in Murtosa municipality is simply due to the fact that this type of building was more available for the conduction of the present study.

The distribution of buildings by interval of year of construction is presented in Figure 2.3. The majority of buildings (71%) were built between the late 19th century and the middle of the 20th century, with more buildings (43%) built in the first half of the 20th century. This distribution is representative of the construction dates of the existing adobe buildings in Aveiro district (Santiago 2007; Silva et al. 2010; Silveira et al. 2013b). Anadia municipality has the only three buildings built after 1950, with the latest built in 1962. It was not possible to ascertain the year of construction of three buildings; however, in view of their characteristics, it is very likely that these buildings were also built in the late 19th century or early 20th century.



[a] There is uncertainty about whether the two buildings in Aveiro were built in the late 19th century or early 20th century.

Figure 2.3: Years of construction and vacancy of the buildings studied.

The distribution of buildings per interval of year of vacancy is also presented in Figure 2.3. Buildings were vacated since the seventies and until the last year of the inspections (i.e. until 2011), with an increase of abandonment in the last two decades. It was not possible to ascertain the year of vacancy of five buildings, but in all five cases vacancy has likely occurred within the time range identified for the other buildings (i.e. from 1970 to 2011). Four buildings were still in use at the time of the visits and analyses conducted.

2.4. Characterisation of facade walls

2.4.1. Structural system

2.4.1.1. Thickness of walls and masonry bonding

All the buildings under study have load-bearing adobe facade walls. In one building (‘*H*33’), adobe masonry is combined with schist masonry. Four buildings (‘*H*24’, ‘*H*25’, ‘*H*37’, and ‘*H*39’) have rear additions that were built at a later stage and are connected to the original structures. The exterior walls of these additions are made with reinforced concrete and ceramic hollow brick masonry and are not analysed in this chapter.

The thickness of the adobe facade walls (including the thickness of the finishing layers) was measured and the results are summarised in Tables 2.3 and 2.4. Considering all the buildings studied, the thickness of the facade walls varies between 0.30 m and 0.71 m. For a clear presentation of the results, the following three cases are distinguished:

i) ‘Case 1’ includes the buildings that have facade walls with constant thickness; this case can be divided in two sub-cases: facade walls with stretcher bond (mean thickness of 0.36 m), and facade walls with other types of bond (with greater thickness, varying from 0.43 m to 0.63 m);
ii) ‘Case 2’ includes the buildings that have facade walls with greater thickness at the semi-basement or ground floor (ranging from 0.45 m to 0.65 m); this case can be divided in two sub-cases: facade walls with stretcher bond at the upper floors (mean

thickness of 0.38 m), and facade walls with other types of bond at the upper floors (only one case, with thickness of 0.50 m);

iii) 'Case 3' includes the buildings that have main facade walls with greater thickness (varying from 0.47 m to 0.71 m) than the other facade walls.

Table 2.3: Thickness of facade walls.

		Thickness of adobe facade walls (m)				
		'Case 1': Constant thickness		'Case 2': Walls with greater thickness at the semi-basement or ground floor		'Case 3': Main facade walls with greater thickness
		Stretcher bond	Other bonds	Ground floor [a] / upper floors (stretcher bond)	Ground floor [a] / upper floors (other bonds)	
Anadia	Mean:	0.35	---	0.45 / 0.35	---	---
	No. of buildings:	6	---	1	---	---
Murtosa	Mean:	0.37	0.53	0.65 / 0.45	---	---
	No. of buildings:	1	5	1	---	---
Aveiro	Mean:	---	0.47	0.50 / 0.35	0.60 / 0.50	[b]
	No. of buildings:	---	2	1	1	3
All municipalities	Mean:	0.36	0.51	0.53 / 0.38	0.60 / 0.50	[b]
	Min.:	0.30	0.43	0.45 / 0.35	---	
	Max.:	0.38	0.63	0.65 / 0.45	---	
	No. of buildings:	7 (33%)	7 (33%)	3 (14%)	1 (5%)	3 (14%)

[a] Or semi-basement.
[b] The thickness of the walls is presented in Table 2.4.

Table 2.4: Thickness of facade walls ('Case 3').

	Thickness of adobe facade walls (m) – 'Case 3': Main facade wall with greater thickness			
	Ground floor	First floor	Second floor	Attic
Building	Main facade / other	Main facade / other	Main facade / other	Main facade / other
'*H*35'	0.50 / [a]	0.50 / 0.35	---	---
'*H*37'	0.71 / 0.48	0.53 / 0.36	---	--- / 0.32
'*H*40'	0.47 / 0.47	0.47 / 0.47	0.47 / 0.35	--- / 0.35
Mean:	0.56 / 0.48	0.50 / 0.39	0.47 / 0.35	--- / 0.34

[a] Impossible to measure.

The majority of the buildings analysed in Anadia municipality (six out of a total of seven) are included in 'Case 1', having facade walls with constant thickness and stretcher bond. The majority of the buildings in Murtosa municipality (five out of a total of seven) are also included in 'Case 1', but have facade walls with greater thickness and other types of bond. Most of the buildings in Aveiro municipality (five out of a total of seven) have facade walls with variable thickness ('Case 2' and 'Case 3'). In general, the buildings analysed in Anadia municipality present the thinnest walls, while the buildings in Murtosa municipality have the thickest walls.

In most buildings, the thinner facade walls were built with stretcher bond (Figure 2.4a). The thickness of these walls corresponds to the width of an adobe brick plus the thickness of the finishing layers. For the thicker facade walls where observation was possible, two different types of bond were identified: English bond (Figure 2.4b) and header bond (Figure 2.4c). English bond appears to be the solution used in one building in Murtosa municipality, and header bond was used in four buildings of Aveiro municipality. In both cases, the thickness of walls corresponds to the length of an adobe brick plus the thickness of the finishing layers. It is possible that other types of bond exist for the thicker facade walls. In particular, the walls with the largest thickness values are possibly composed of a double leaf system, combining a stretcher bond leaf and a header bond leaf; however, it was not possible to confirm this hypothesis.

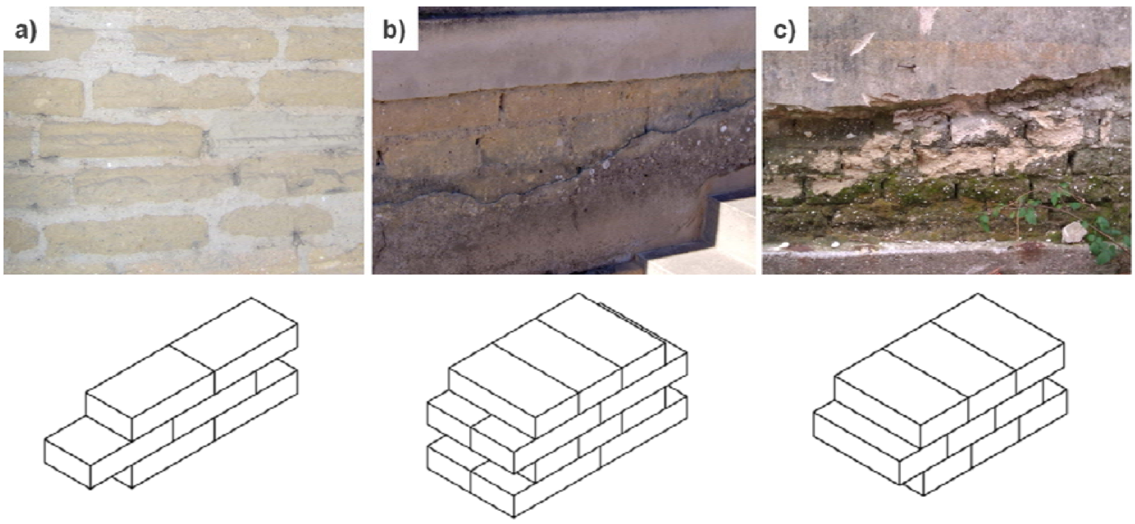


Figure 2.4: Types of adobe masonry bond: a) stretcher bond; b) English bond; c) header bond.

2.4.1.2. Wall openings

It was possible to observe the structure of the facade wall openings in eight out of the total of twenty-one buildings studied (i.e. in 38% of the buildings). The common solution consists of a wooden lintel positioned above the opening with its ends embedded in the adobe masonry (Figures 2.5a, 2.5b, and 2.5c). In some cases, other reinforcing elements, such as small ceramic elements or stones, were added sparsely to the masonry adjacent to the sides of the openings (Figure 2.5b). In two buildings, the wooden lintel is combined with other elements:

i) In building '*H*24', in Anadia municipality, two adobe bricks are positioned diagonally above the wooden lintel, forming a triangle (Figure 2.5c);
ii) In building '*H*28', in Murtosa municipality, an arch made with ceramic elements is located above the wooden lintel (Figure 2.5d); in this building the wooden lintel extends along the perimeter of the building.

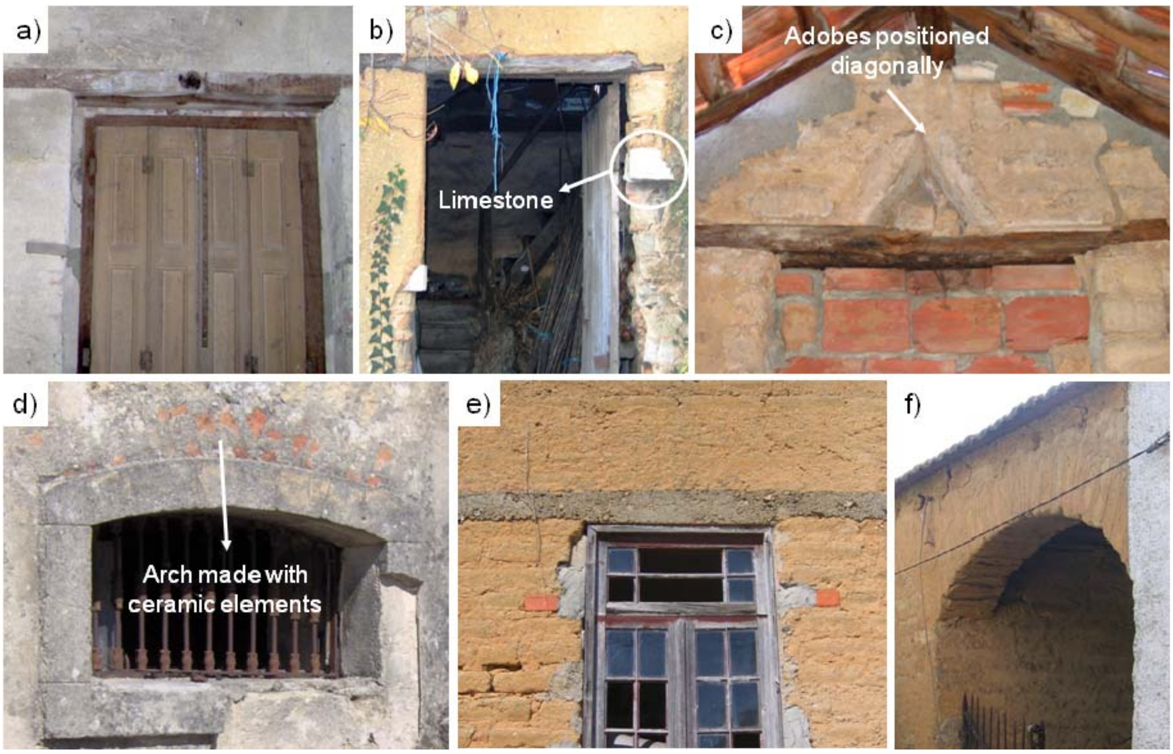


Figure 2.5: Structural system of facade wall openings.

In buildings '*H*23' and '*H*27', located in Anadia municipality, in place of the wooden lintel there is a reinforced concrete beam that runs along the perimeter of the building

(Figure 2.5e). In building ‘*H*27’, the openings are sometimes laterally reinforced with cement mortar.

Five buildings have a small number of wide openings built with arches. In four buildings the arches are made with adobe bricks (Figure 2.5f) (in one case combined with a wooden lintel), and in one building the arch is made with small solid ceramic bricks.

2.4.1.3. Connection between walls

It was possible to observe the connection between facade walls in thirteen out of the total of twenty-one buildings (i.e. in 62% of the buildings). In these buildings, the connection between facade walls is made by the interlocking of adobe bricks in the corners (Figure 2.6a). In a few cases, small ceramic bricks or stones are sparsely distributed in the corners for additional reinforcement (Figure 2.6b).

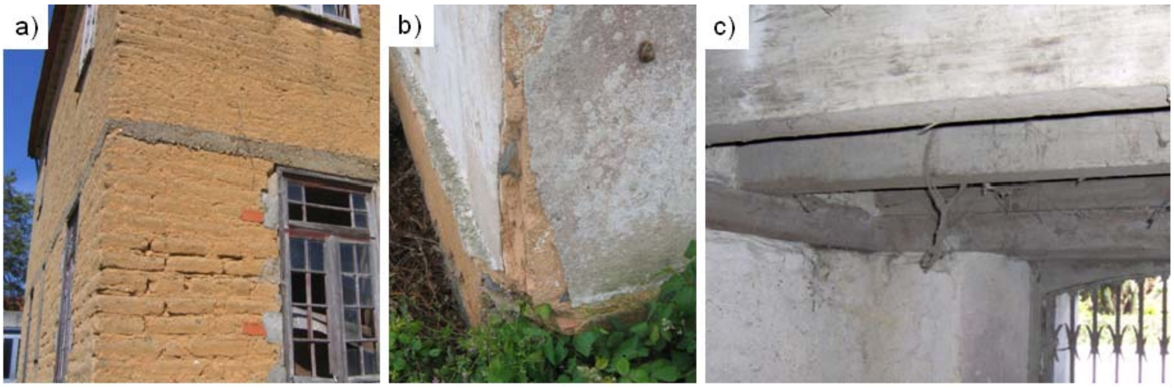


Figure 2.6: Connection between facade walls.

Five buildings, out of a total of nine where observation was possible, have bond beams (Table 2.5) – a bond beam is a structural element that runs continuously along the perimeter of the building, tying the facade walls together and contributing to the overall stability of the building. Three buildings in Anadia municipality and one building in Aveiro Municipality have thin reinforced concrete bond beams (Figure 2.6a). In the ground floor (or semi-basement), the bond beams were observed at the level of the first floor structure (in buildings ‘*H*23’, ‘*H*25’, and ‘*H*39’) or immediately above the openings, working as a lintel in these openings (in building ‘*H*27’). In buildings ‘*H*23’ and ‘*H*27’, in the first floor, the bond beams were observed above the openings (working also as a lintel in these openings). In buildings ‘*H*25’ and ‘*H*39’, it was not possible to confirm the existence of a second bond beam at the top of the first floor. Building ‘*H*28’, located in Murtosa municipality, has a wooden bond beam at the level of the first floor structure, which also works as a lintel in the openings of the ground floor (Figure 2.6c), and a second wooden bond beam above the openings of the first floor, working also as a lintel in these openings. It was not possible to check the nature and quality of the connections of the wooden beams in the corners – these beams must be effectively interconnected in order to contribute to the overall stability of the building.

Table 2.5: Types of bond beam observed in the buildings studied.

		Bond beams			
		Wood	Reinforced concrete	Without bond beams	Not observed
No. of buildings	Anadia	0	3	3	1
	Murtosa	1	0	1	5
	Aveiro	0	1	0	6
	Total	1 (5%)	4 (19%)	4 (19%)	12 (57%)

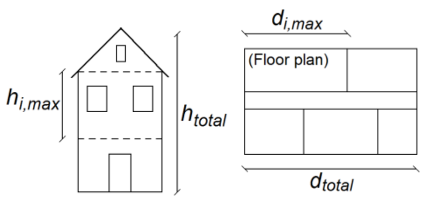
2.4.1.4. Dimensions and slenderness ratios of walls

The maximum total height of facade walls, inter-storey height, total length of facade walls, and distance between lateral supports (i.e. transverse walls) are presented in Table 2.6. The maximum ratio between the inter-storey height and the corresponding thickness of walls, and the maximum ratio between the distance between lateral supports and the corresponding thickness of walls are also presented. For the sake of simplicity, both ratios will be referred to as ‘slenderness ratios’. The limits indicated in different technical standards for some of these parameters are also presented in Table 2.6. The standards considered are: ‘NZS 4297:1998 Engineering design of earth buildings’ (SNZ 1998a), with indications for earthen construction, in general; ‘Norma técnica de edificación NTE E.080 Adobe’ (ICG 2006) and ‘International Building Code (IBC)’, with recommendations for adobe construction; and ‘Eurocode 8’, with indications for ‘simple masonry buildings’.

Table 2.6: Dimensions and slenderness ratios of facade walls.

		Adobe facade walls					
		Maximum height (m)		Maximum length (m)		Maximum slenderness ratio	
		Total (h_{total} [a])	Inter-storey ($h_{i,max}$ [a])	Total (d_{total} [a])	Between transverse walls ($d_{i,max}$ [a])	$(h_i / e)_{max}$ [b]	$(d_i / e)_{max}$ [b]
Anadia	Mean:	7.0	3.4	11.4	6.3	9.6	17.1
Murtosa	Mean:	7.9	3.9	17.0	9.5	8.1	19.5
Aveiro	Mean:	7.9	4.0	16.2	9.4	9.9	21.1
All municipalities	Mean:	7.6	3.8	14.9	8.4	9.2	19.2
	Min.:	3.5	2.8	8.5	3.9	7.0	10.3
	Max.:	12.1	4.8	32.3	13.8	14.8	28.0
	CV:	34%	15%	40%	37%	19%	27%
Standard limits	‘NZS 4297’ (SNZ 1998)	---	---	---	---	≤ 6 [c]	---
	‘NTE E.080’ (ICG 2006)	---	---	---	---	≤ 6 [d]	≤ 12
	‘IBC’ (ICC 2009)	---	---	---	≤ 7.315	≤ 10	---
	‘Eurocode 8’ (CEN 2010)	---	---	---	≤ 7	---	---

[a] Schematic explanation of notation:



[b] ‘e’: wall thickness.

[c] Considering the earthquake zone factor for the areas of greater seismic hazard in New Zealand.

[d] Considering the existence of a bond beam; if $(h_i / e)_{max} > 6$, additional reinforcement is required.

Both load-bearing interior walls (normally made with adobe) and non-load-bearing interior walls (normally *tabique* walls, made with a wooden structure filled and coated with lime mortar) were considered when determining the maximum distance between transverse walls. Even though *tabique* walls were generally built simply as partition walls, they have some load-bearing capacity and may improve the lateral restraint of facade walls. In the calculation of the slenderness ratios, it was also assumed that the connections between perpendicular walls and between facade walls and floor and roof structures are effective. However, considering the observations carried out in situ, it can be concluded that these connections may not always be effective, especially between facade walls and *tabique* walls and also between facade walls and floor and roof structures. Thus, in some cases, the real slenderness ratios of the facade walls are likely greater than the ratios calculated.

The maximum inter-storey height of the buildings studied shows little variation (CV = 15%), with a mean value of 3.8 m. The maximum total height and total length of facade walls and the maximum distance between transverse walls exhibit greater variability, with coefficients of variation ranging up to 40%, and with mean values of 7.6 m, 14.9 m, and 8.4 m, respectively. The slenderness ratio calculated using the inter-storey height has a mean value of 9.2, with a coefficient of variation of 19%. The mean slenderness ratio calculated using the distance between lateral supports is 19.2, with a coefficient of variation of 27%.

In general, the buildings analysed in Anadia municipality have facade walls with lower height and lower distances between lateral supports, when compared to those of other municipalities. However, given that the facade walls of the buildings are generally thinner, there is no great difference between the slenderness ratios calculated for the buildings of this municipality and those of the other municipalities.

By comparing the results obtained with the limits indicated in different technical standards, the following observations can be made:

- In general, the maximum distance between lateral supports of facade walls is greater than the maximum limits indicated in ‘IBC’ (ICC 2009) and ‘Eurocode 8’ (CEN 2010);
- For all the buildings studied, the slenderness ratio calculated using the inter-storey height is greater than the maximum limit indicated in ‘NTE E.080’ (ICG 2006) and ‘NZS 4297’ (SNZ 1998);
- All the buildings analysed in Murtosa municipality, four buildings in Anadia municipality, and four buildings in Aveiro municipality have facade walls with slenderness ratios (calculated using the inter-storey height) that respect the limit indicated in ‘IBC’ (ICC 2009);
- Only one building in each of the three municipalities respects the limit indicated in ‘NTE E.080’ (ICG 2006) for the slenderness ratio calculated using the distance between lateral supports.

It can thus be concluded that, in general, the limits indicated in existing standards for the distance between lateral supports and slenderness ratios of facade walls are not respected in the buildings under study. This may lead to instability of the walls, particularly when subjected to horizontal loads, such as those imposed by the roof structure or seismic loads. It is important to note that Aveiro district is located in an area of moderate seismic hazard (CEN 2010), while Peru and New Zealand, countries for which two of the standards used were created, are regions of high seismic hazard. Nevertheless, the seismic demand on existing structures in Aveiro may be significantly amplified for the soft foundation soils that are common in this region (CEN 2010; Bonito 2008), and thus seismic loads should be considered carefully in the assessment and rehabilitation of existing buildings.

2.4.1.5. Foundation system

The buildings studied were built with strip foundations, both for the support of the facade walls and interior adobe walls. In five buildings, the foundation of the facade walls was made with stone masonry and, in two buildings, with adobe masonry (Table 2.7). In the majority of the buildings (67%), however, it was not possible to observe the type of material used in the foundation. It is important to note that, according to existing studies, adobe masonry was more commonly used than stone masonry in the foundations of adobe buildings (Santiago 2007; Maia 2009; Tavares 2009).

Table 2.7: Types of masonry used in the foundation of facade walls.

		Foundations		
		Stone masonry	Adobe masonry	Not observed
No. of buildings	Anadia	3	1	3
	Murtosa	2	0	5
	Aveiro	0	1	6
	Total	5 (24%)	2 (10%)	14 (67%)

The stone foundations observed rise to a height above the ground level ranging approximately from 0.20 m to 0.80 m. The stones are irregular, with varied sizes and shapes, and are generally bonded with an earth mortar (with or without lime). In buildings '*H*21', '*H*22', and '*H*25', in Anadia municipality, the foundations were made with limestone (Figure 2.7a); in building '*H*33', in Murtosa municipality, schist was used

(Figure 2.7b); and in building ‘*H*34’, in Murtosa municipality, a combination of schist and red sandstone (‘Eirol stone’) was adopted (Figure 2.7c). Given that ‘mud adobes’ degrade very easily in contact with water, buildings with facade walls made with ‘mud adobes’ (described in subsection 2.4.3), such as ‘*H*33’ and ‘*H*34’, in Murtosa municipality, required the use of stone or ‘lime adobe’ foundations.

Figure 2.7: Types of stone observed in the foundation of facade walls: a) limestone; b) schist; c) schist and red sandstone.

With the inspection carried out, it was possible to identify the type of material used but not the dimensions, defects, or state of conservation of the foundations. Some defects observed in the facade walls, however, may be caused by differential foundation settlement, as will be further discussed in subsection 2.5.1.

2.4.2. Exterior wall finishing solutions

The different types of wall finishing solutions observed on the outer surface of the facade walls are presented in Figure 2.8. The facade walls of the majority of the buildings (62%) are rendered with lime mortar (Figure 2.9a). This was the solution traditionally used in the adobe buildings of Aveiro district. In some of these buildings, cement mortar was later applied in small areas of the wall to cover existing lime mortar deterioration. In 33% of the buildings studied, the facade walls are entirely coated with a more recent layer of cement mortar, normally applied over the existing layer of lime mortar (Figure 2.9b).

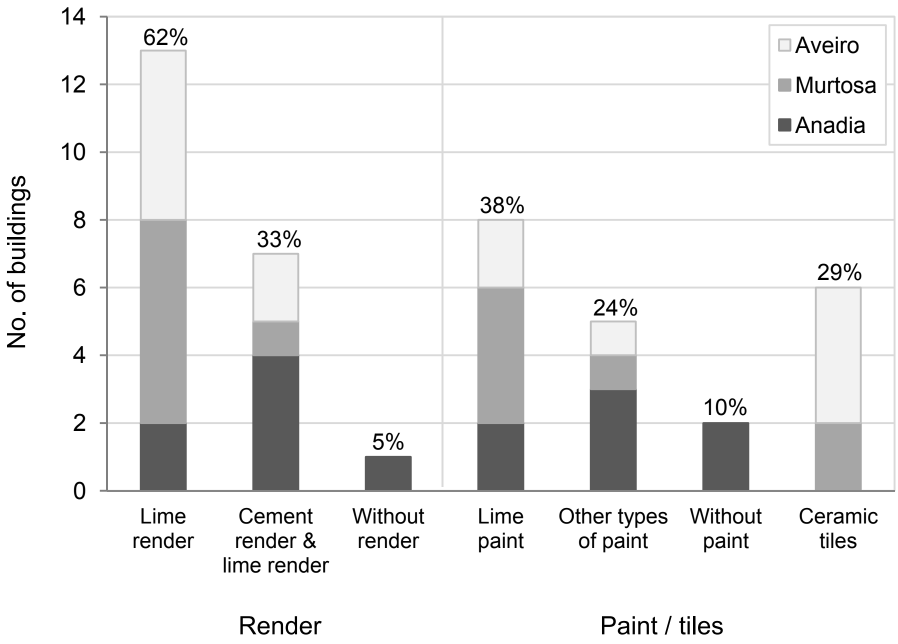


Figure 2.8: Exterior wall finishing solutions.

Figure 2.9: Exterior wall finishing solutions: a) lime render and lime paint; b) cement render; c) ceramic tiles.

A large percentage of buildings (76%) have a thicker layer of mortar at the base of the facade walls for added protection against the action of rainwater. This solution was adopted in almost all buildings of Murtosa and Aveiro municipalities but only in three buildings of Anadia municipality. In some buildings, this solution was used only on the main facade wall or on the facade walls that are visible from the street. In buildings ‘*H*35’ and ‘*H*40’, located in Aveiro city, this protection is made with a layer of limestone and granite, respectively.

The facade walls of 38% of the buildings studied are finished with lime paint (applied over the lime render) (Figure 2.9a), which was a solution traditionally used in the adobe buildings of Aveiro district. Other types of paint, which are now commonly used and generally have impermeable characteristics, were observed in 24% of the buildings. These different types of paint were added later, sometimes directly over the existing layers of lime paint. In building '*H*27', located in Anadia municipality, the walls were not rendered. In building '*H*21', also in Anadia Municipality, the facade walls were rendered but not painted.

Ceramic tiles, which were also a finishing solution traditionally used in Aveiro district, were observed in the facade walls of two buildings located in Murtosa municipality and four buildings in Aveiro municipality (i.e. in 29% of the buildings studied) (Figure 2.9c). Tiles were applied over the lime render layer and were generally used in the facade walls that could be observed from the street. In the buildings of Aveiro municipality, in particular, ceramic tiles are only used in the main facades. In these buildings, the walls that are not finished with tiles are finished either with lime paint or with a more recent layer of a different type of paint.

Anadia municipality has the largest number of buildings studied that were subject to recent interventions, and thus the facade walls of these buildings frequently have recent layers of cement render and paint with impermeable characteristics. In Murtosa and Aveiro municipalities, many of the buildings that were analysed have facade walls that were not subject to any recent interventions, remaining with the original finishing solution (i.e. lime render with lime paint or ceramic tiles).

2.4.3. Traditional masonry materials

The dimensions of the adobes and the thickness of the traditional mortars (i.e. the mortars made with lime and sand or with clayey soil) used in the facade walls of the buildings under study are presented in Table 2.8. The number of buildings where it was possible to perform measurements is also indicated. The dimensions of the adobes are relatively uniform throughout the different regions. Considering all municipalities, the mean dimensions of adobes are: 0.45 x 0.31 x 0.11 m^3. The thickness of render and plaster is significantly more variable, even for the same building. Considering all municipalities, the mean joint thickness is approximately 0.03 m and the mean render and plaster thickness is approximately 0.02 m. There is a tendency for the render (applied on the outer surface of facade walls) to be slightly thicker than the plaster (applied on the inner surface of facade walls).

Table 2.8: Dimensions of adobes and thickness of joint, plaster, and render.

		Adobe dimensions (m)			Mortar thickness (m)		
		Length	Width	Height	Joint	Plaster (interior)	Render (exterior)
Anadia	Mean:	0.45	0.32	0.11	0.028	0.018	0.023
	n [a]:	6	5	6	6	5	4
Murtosa	Mean:	0.46	0.32	0.11	0.029	0.025	0.022
	n [a]:	5	4	5	5	4	4
Aveiro	Mean:	0.43	0.30	0.12	0.028	0.013	0.025
	n [a]:	4	4	5	5	3	1
All municipalities	Mean:	0.45	0.31	0.11	0.028	0.019	0.023
	Min.:	0.41	0.25	0.09	0.020	0.005	0.008
	Max.:	0.53	0.37	0.14	0.038	0.030	0.035
	CV:	7%	11%	13%	17%	47%	41%
	n [a]:	15	13	16	16	12	9

[a] Number of buildings where measurement was possible.

The great majority of the buildings studied (eighteen out of a total of twenty-one buildings, i.e. 86%) have facade walls made with ‘lime adobes’ (Figure 2.10a). Two buildings (‘*H*33’ and ‘*H*34’) have facade walls made with ‘mud adobes’ (Figure 2.10b), and in one building (‘*H*26’) a combination of both types of adobes was used. As explained in Chapter 1, in Aveiro district, at an early stage, adobes were made with clayey soil (‘mud adobes’), sometimes with the addition of natural fibres, such as straw (Oliveira and Galhano 1992; Santiago 2007). From the middle of the 19th century, adobes stabilised with lime (‘lime adobes’) progressively became the solution commonly used. These adobes were made with arenaceous soil with a small silt-clay fraction (Santiago 2007) and air-lime in a percentage that normally ranged between 25% and 40% (Teixeira and Belém 1998). Local materials were used in the production of both types of adobes.

The mortar used in the joints of the ‘mud adobe’ facade walls under study has a similar composition to that of the adobes (i.e. it is made with clayey soil) (Figure 2.10b).

The traditional mortar used for renders and plasters, in this type of wall, is made with arenaceous soil and air-lime (Figure 2.10b). The mortar used in the 'lime adobe' walls – for joints, plasters, and renders – is also made with arenaceous soil and air-lime (Figure 2.10a). In some buildings, the render consists of just one layer of mortar with composition similar to that of the lime adobes. In other buildings, it includes one or two additional thinner layers of mortar made with finer sand and a greater percentage of lime.

Figure 2.10: Traditional masonry materials.

In buildings '*H*33' and '*H*34', both with 'mud adobe' facade walls, small pieces of stone or ceramic material were also used in the joints (Figure 2.10b), contributing to increase the strength of the walls and the adhesion of the render to the walls. The vertical joints in building '*H*34', in particular, are filled with pieces of stone and ceramic material and are approximately 0.10 m thick. Since this large thickness value is an exception, it was not considered in the analysis presented in Table 2.8.

The fact that buildings with facade walls made entirely with 'mud adobes' were only found in Murtosa municipality is in agreement with the analysis of Santiago (2007). Santiago (2007) noted that, at present, 'mud adobes', although rare, are more commonly found at the north of Vouga river (as is the case of Murtosa municipality) and that, at the south of this river, the production with this type of adobe must have been abandoned for longer. On the other hand, the two buildings made with 'mud adobes' ('*H*33' and '*H*34') are among the oldest buildings studied – and, considering their characteristics, are probably the oldest –, which also justifies the use of 'mud adobes'.

2.5. Defects in facade walls

The most common defects observed in the facade walls of the buildings studied are presented in Figure 2.11. The number and percentage of buildings, per municipality, that present each defect are also indicated. The most common defects observed and their possible causes are described below.

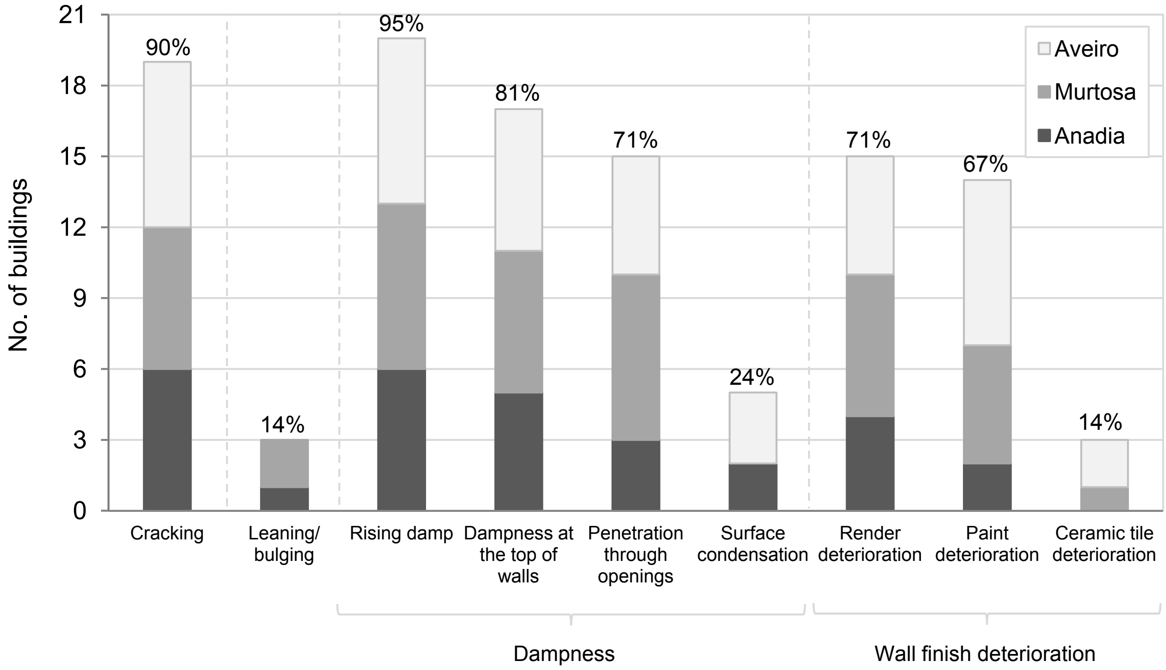


Figure 2.11: Common defects observed in facade walls.

2.5.1. Cracking and partial collapse

Cracking of facade walls was observed very frequently in the inspections carried out. In fact, 90% of the buildings studied have facade walls with superficial and structural cracks (Figure 2.11). The most relevant types of cracking observed are represented schematically in Figure 2.12. The number of buildings, per municipality, that suffer from each defect, is presented in Figure 2.13. For each type of cracking, the percentage of buildings, in relation to the total number of buildings studied, is also indicated. The different types of cracking observed and their possible causes are described below. A brief analysis of the damage that led to the partial collapse of two facade walls in building ‘*H*34’ is also presented.

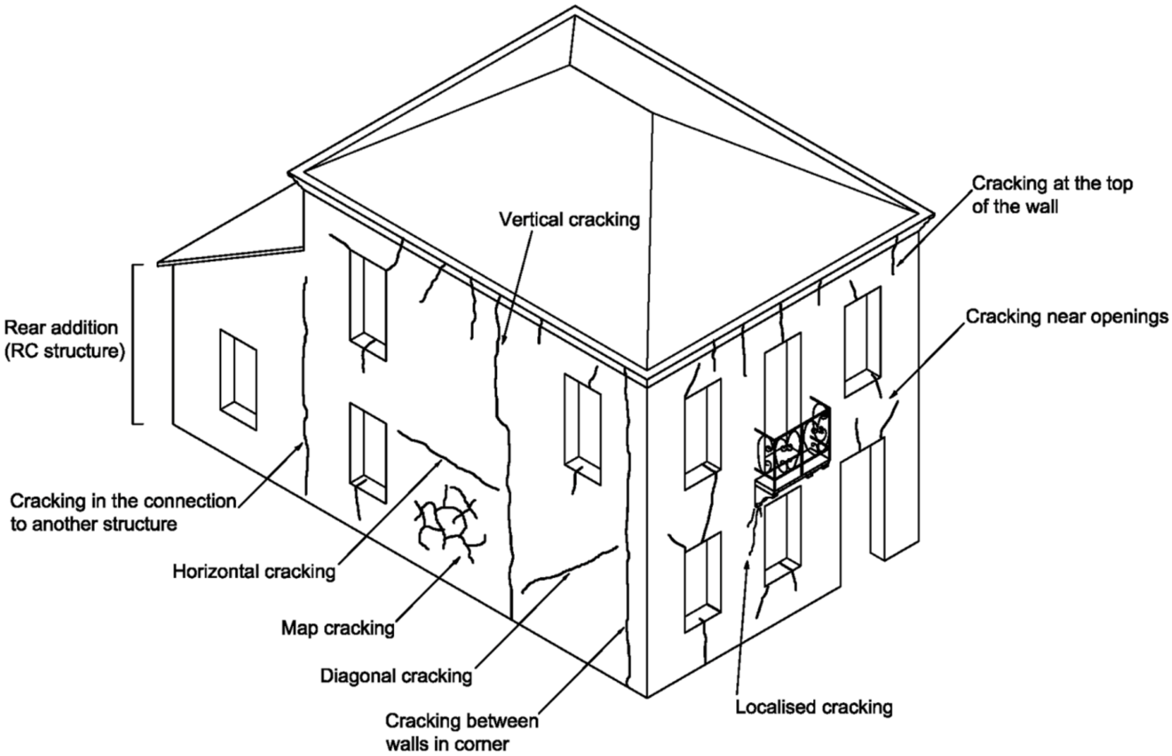


Figure 2.12: Schematic representation of the types of cracking observed in facade walls.

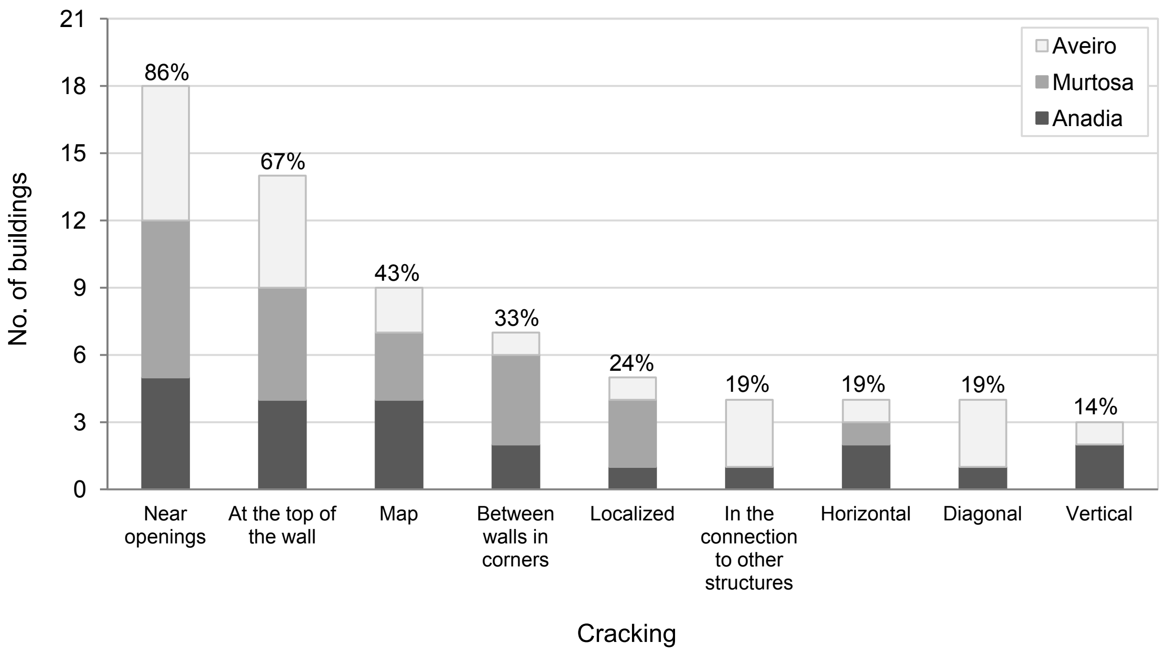


Figure 2.13: Types of cracking observed in facade walls.

2.5.1.1. Cracking near openings

The cracks near openings observed in the buildings studied are mainly located above or below the openings, frequently near the corners, and generally have vertical or diagonal orientation (Figures 2.12 and 2.14). In some cases, cracking is thin and superficial, affecting only the render layer; more frequently, however, it is thick and deep, affecting also the support structure. In one building (‘*H*21’), intense cracking led to the partial disintegration of masonry above two openings (Figure 2.14). Common causes for cracking near openings are: excessive load (Thomaz 2003), generally imposed by the roof or floor structures; insufficient support of the masonry above the openings; stress concentration in the corners; and excessive percentage of wall area with openings. In some of the cases observed, differential foundation settlement is also a possible cause for the existing cracking (Richardson 2001; Thomaz 2003).

Figure 2.14: Cracking near openings.

2.5.1.2. Cracking at the top of the wall

The cracking observed at the top of the facade walls of the buildings studied is mainly vertical or diagonal (Figures 2.12 and 2.15). In some cases the cracks are thin and superficial, but in other cases the cracks are thick and affect the support structure. This type of cracking is generally caused by excessive load or deformation imposed by the roof structure (Thomaz 2003).

Figure 2.15: Cracking at the top of walls.

2.5.1.3. Map cracking

Map cracking is a network of thin and superficial cracks, oriented in a random pattern (Figure 2.12). It generally occurs as the result of the drying shrinkage of mortar (Marshall et al. 2014). This type of cracking was mainly observed in cement render, but a few cases in lime render were also identified.

2.5.1.4. Cracking between perpendicular walls in corners

Structural cracking between perpendicular facade walls, in corners, was observed in some of the buildings under study (Figures 2.12 and 2.16). This type of cracking is generally vertical, following along the area of connection between walls. In some of the cases observed, vertical cracking is combined with diagonal or scattered cracking along the corner. Possible causes for this type of cracking are: deformation of the walls due to variations in temperature or moisture content (Thomaz 2003); differential foundation settlement (Pagaimo 2004); and horizontal thrust imposed by the roof which leads to the rotation of one or both perpendicular walls (Pagaimo 2004). These factors can lead to cracking or even separation between facade walls, especially when combined with a weak connection between walls.

Figure 2.16: Cracking between perpendicular walls in corners.

2.5.1.5. Localised cracking

Localised cracking in the facade walls, due to stress concentration, was observed near the points of support of balconies (Figures 2.12 and 2.17a), steel rods, and beams. This type of cracking is generally thick, affecting the support structure. The cracks are vertical or diagonal and are sometimes combined with crushing around the support points.

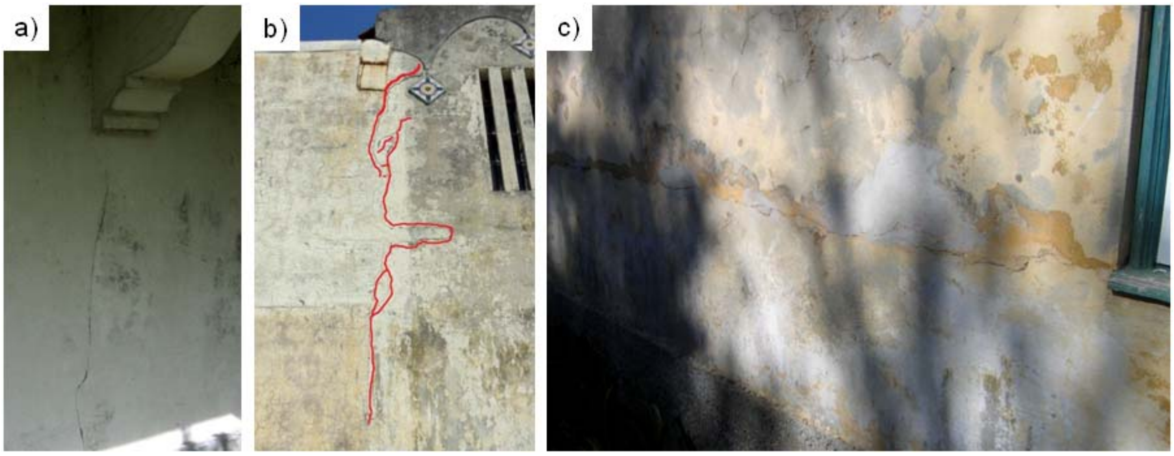


Figure 2.17: a) Localised cracking near the support of balcony; b) cracking in the connection to rear addition; c) long horizontal cracking.

2.5.1.6. Cracking in the connection of buildings to other structures

Cracking was observed in the connection of buildings to adjacent land dividing walls and in the connection of the original buildings to rear additions (Figures 2.12 and 2.17b).

In these cases, cracks are generally vertical or diagonal, following along the connection surface. Possible causes for this type of cracking are the differential movement of the structures due to variations in temperature or moisture content (Richardson 2001) and differential foundation settlement (Thomaz 2003). In the buildings with rear additions, in particular, the adjoining constructions, made with different materials and structural solutions, behave differently, which naturally leads to cracking in the interface.

2.5.1.7. Horizontal cracking

Long horizontal cracking was observed in the facade walls of some of the buildings studied (Figures 2.12 and 2.17c). In most cases, the cracks are thick and may affect the support structure. A possible cause for long horizontal cracking is excessive load imposed by the roof structure, causing flexion of the wall (Thomaz 2003). However, in the cases observed, the horizontal cracks may be the result of cracking that started in areas of structural fragility (such as near openings or near collapsed areas) and that extended horizontally along the bed joints. In one building (‘*H*23’), horizontal cracking is located at the level of the first floor concrete bond beam and is likely the result of incompatibility between the behaviour of two different materials (adobe masonry and concrete).

2.5.1.8. Diagonal cracking

Long diagonal cracking was also observed in the facade walls of some of the buildings studied (Figures 2.12 and 2.18a). In all cases, the cracks are thick and appear to affect the support structure. In building ‘*H*39’, one diagonal crack is located near an opening, extending diagonally and downward towards a corner of the building, and seems to be caused by excessive load imposed by the roof structure. In the other cases where this type of cracking is observed, cracks appear to be caused by differential foundation settlement – in these cases, the diagonal cracks lean towards the point where the largest settlement occurred (Richardson 2001; Thomaz 2003). In building ‘*H*39’, one diagonal crack of this type is particularly severe, compromising the structural stability of the building (Figure 2.18a).

Figure 2.18: a) Diagonal cracking; b) vertical cracking in the central area of the wall.

2.5.1.9. Vertical cracking in the central area of the wall

Vertical structural cracking in the central area of the facade walls was observed in a few of the buildings under study (Figures 2.12 and 2.18b). The vertical cracks are long, sometimes extending from the top to the bottom of the walls. Possible causes for this type of cracking are (Thomaz 2003): deformation of the walls due to variations in temperature or moisture content; differential foundation settlement; and excessive concentrated load imposed by the roof structure.

2.5.1.10. Partial collapse

One of the buildings studied (‘*H*34’) has two facade walls where intense cracking led to the collapse of part of the walls. In the right side facade, the section of the wall above an arch made with adobe and with a large span (3.75 m) collapsed. In the rear facade, there was a section of wall with reduced thickness, made with ceramic bricks – covering a large niche, in a room that used to function as a chapel –, that also collapsed.

2.5.1.11. Final comments

The main possible causes of the cracking observed in the facade walls studied are: deformation of the walls due to variations in temperature or moisture content; differential foundation settlement; and excessive load imposed by the roof structure. The differential settlement of the foundations can be due to a combination of different factors, such as: variation of soil moisture; non-uniform loading; soil heterogeneity; soil consolidation; and different consolidation of fill soils. In some areas of Aveiro district, existing highly compressible and low strength soils (Bonito 2008) may contribute to the foundation settlement problems observed. It is important to note that cracking of structural masonry is generally the result of a complex combination of factors, and thus it is often difficult to isolate specific causes.

Since excessive load imposed by the roof structure is one of the main possible causes of existing cracking, it is important to briefly describe the type of roof structure of the buildings studied. All the buildings, with the exception of building '*H*23' (which has a metal roof structure), have wooden roof structures. Two types of structure were observed: with beams or with trusses as the main support elements. The majority of the buildings have roof structures made with king post trusses. These trusses are closed, i.e. have a horizontal beam (tie beam) that ties together the feet of the opposite rafters – this type of structure tends to reduce the horizontal thrust exerted by the roof on the facade walls. In all the roofs, the ends of the main structural elements are embedded in the facade walls, normally without additional reinforcement of the areas of connection – which, in some cases, leads to cracking and crushing around these areas. It was not observed a clear correlation between the type of roof structure and the cracking observed in the facade walls of the buildings studied.

The possible causes identified, combined with the fact that adobe masonry is characterised by low tensile and shear strength and brittle behaviour – as will be seen in later chapters (Silveira et al. 2012; Silveira et al. 2015) –, lead to significant cracking in the facade walls of the buildings studied. Walls made with 'mud adobes', in particular, have very low strength values and thus are more vulnerable to cracking and collapse processes. Indeed, the three buildings where 'mud adobes' were used ('*H*26', '*H*33' and '*H*34') are among the buildings with the most severe cracking defects. The fact that the facade walls of the buildings studied are excessively slender − when compared to the slenderness ratios recommended in technical standards − may contribute to their instability, especially when submitted to horizontal loads, such as seismic loads or loads imposed by the roof structure, which can also result in structural cracking.

In some of the cases studied, the path of structural cracks is influenced by the position of masonry joints. In these cases, cracks follow along mortar joints in part of their path, sometimes displaying a stepped pattern. When following along mortar joints, cracks are mainly located in the interface between adobe bricks and joint mortar. In the adobe masonry traditionally used in Aveiro district, the strength of adobe is generally close to the strength of mortar; thus, cracks that follow along mortar joints are usually due to insufficient bond between the two materials, as will also be seen in Chapter 5 (Silveira et al. 2015).

Overall, the cracking observed in the facade walls of the buildings studied can severely compromise their structural integrity. In addition, cracking creates areas of vulnerability to water seepage that can further degrade adobe masonry. Effective measures to address and prevent cracking in the facade walls of existing adobe buildings are therefore fundamental.

2.5.2. Leaning or bulging of walls

Three out of the twenty-one buildings studied have leaning or bulging facade walls (Figure 2.11). These defects, however, are not very pronounced. Excessive vertical load imposed by the roof (USDOI 1978) or floor structures, expansion of the soil below the foundation, and horizontal thrust exerted by the roof structure (Pagaimo 2004), combined with excessive slenderness of walls and weak connection between walls and other structural elements, are possible causes for these defects.

2.5.3. Dampness

Dampness problems were observed in almost all the buildings studied (Figures 2.11 and 2.19). The different dampness problems identified and their main causes are presented as follows.

Figure 2.19: a) Rising damp; b) dampness at the top of walls; c) water penetration through window openings; d) surface condensation.

2.5.3.1. Rising damp

Rising damp is a very common defect (Figure 2.19a), observed both in the buildings with adobe foundations and in the buildings with stone foundations. The signs of rising damp generally consist of damp patches, mould, moss, mortar detachment, and paint peeling located at the bottom of the facade walls. In some buildings, these signs are only visible on the outer surface of walls. In these cases, surface water may be the main cause of rising damp (Freitas et al. 2008). In other buildings, the signs of rising damp are also visible on the inner surface of walls and are of equal or even higher intensity in these surfaces. In these cases, groundwater may be the main cause of rising damp (Freitas et al. 2008).

Several factors appear to contribute to the problem of rising damp observed in the buildings studied. Adobe is a material with high capillarity (Martins 2009; Coroado et al. 2010) and thus adobe walls are very susceptible to this phenomenon. In addition, many of the buildings studied do not have a rainwater drainage system or have a malfunctioning drainage system, which causes the accumulation of water near the base of the buildings. In some cases, dense vegetation near the wall hinders rainwater drainage, causing its retention at the base of the wall. The existence of structures, like neighbouring buildings or land dividing walls, that frequently or permanently shade the lower part of the wall also contribute to this problem by slowing down the evaporation process. For a similar reason, it was observed that facade walls with northern orientation have more rising damp problems.

2.5.3.2. Dampness at the top of walls

Dampness at the top of walls is usually the result of water penetration through the roof. This problem is generally observed on the inner surface (Figure 2.19b) and sometimes on the outer surface of walls. On the inner surface, damp patches, frequently combined with dripping water stains, are more intense at the top of the walls and often extend to the bottom. In some buildings, the formation of mould in these areas can be observed. On the outer surface of walls, the signs of dampness generally consist of mould and, in some cases, of peeling paint. In some buildings, the eaves have deficiencies (such as missing tiles or cracked tiles) that further contribute to this problem. The fact that most roofs have a lower slope at the base may lead to the accumulation of water in this area, which can contribute to the existing penetration problems. In addition, the formation of vegetation is this area, visible in many of the buildings studied, contributes to the accumulation of water and also to the damage of roof tiles.

Three buildings (‘*H*38’, ‘*H*39’ and ‘*H*40’, in Aveiro municipality) have roof parapets on the main facade walls. These facade walls have dampness problems, visible on the parapets and at the top of the walls in the interior of the buildings. In these cases, the roof parapet acts as a barrier to water drainage, leading to moisture problems in the roof and facade walls (Tavares et al. 2012).

In some cases, the wooden lintels located above openings also have dampness problems caused by the rainwater that flows from the roof or hits the walls directly. Due to the prolonged presence of dampness, these elements suffer from biological deterioration, rotting, cracking, and fracture. The existing degradation in some cases may progress to a point where the lintel no longer fulfils its structural function.

Another type of defect observed in some buildings and included in this category is dampness on the outer surface of facade walls near the roof of contiguous lower-height building sections. Deficiencies in the drainage of rainwater from the adjacent roof lead to the accumulation of rainwater near the facade wall of the higher building section. Signs of this problem consist mainly of mould, sometimes combined with peeling paint and render detachment.

2.5.3.3. Water penetration through window openings

Rainwater penetration through window openings is also a common cause of dampness in the facade walls of the buildings studied (Figure 2.19c). Rainwater seeps through the connection between window frames and facade walls or through gaps in the window frames. This problem is generally manifested by the presence of damp patches, dripping water stains, and peeling paint on the area below the openings.

2.5.3.4. Surface condensation

Another dampness problem that was identified, but which is less common and usually not very intense, is surface condensation (Figure 2.19d). This defect occurs on the inner surface of the facade walls of some of the buildings studied and is manifested by the presence of mould. This problem generally occurs in facade walls with northern orientation and in bathrooms. In some cases, it is distributed along the wall and, in other cases, it is concentrated at the top of the wall or is markedly more intense in this area. There is also a tendency for this problem to occur at the corners of walls.

The fact that earth has better breathability (i.e. water vapour permeability, hygroscopicity, and capillarity) than most modern construction materials (May 2005; Jaquin 2009) and other factors, such as the existence of gaps in traditional window and door frames, which allow the flow of air throughout the building, and the existence of high ceilings in some buildings, may help explain the relatively small number of buildings (five out of a total of twenty-one, i.e. 24%) where surface condensation was observed.

2.5.3.5. Final comments

The main causes of the dampness problems observed in the facade walls of the buildings studied are the lack or malfunction of the rainwater drainage systems and the existence of deficiencies in the roof and window openings that lead to rainwater penetration. These deficiencies, combined with the fact that adobe is very vulnerable to the action of water, result in defects that, if not timely and adequately addressed, compromise the habitability of buildings and may even jeopardise their structural integrity – the presence of water in the walls for extended periods of time may lead to the disintegration and degradation of adobe masonry, which can affect the structural performance of walls.

2.5.4. Exterior wall finish deterioration

In addition to the cracking and dampness defects described above, the exterior wall finishes of a significant percentage of the buildings studied suffer from other deterioration problems (Figures 2.11 and 2.20). These defects and their respective causes are described below.

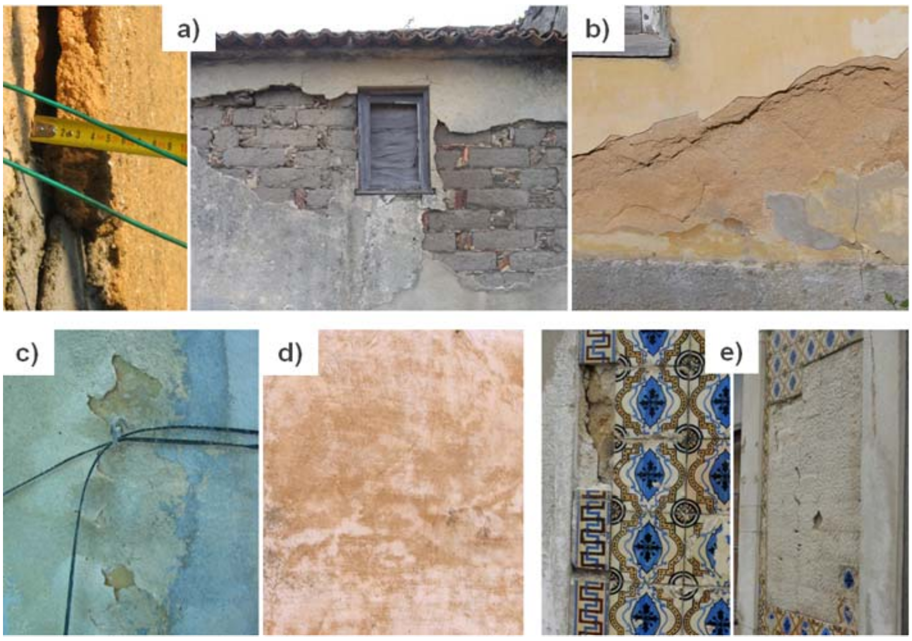


Figure 2.20: a) Render detachment; b) render erosion; c) paint peeling; d) paint erosion; e) ceramic tile deterioration.

2.5.4.1. Render deterioration

Render deterioration was observed in 71% of the buildings studied. Two main forms of render deterioration were identified: detachment and erosion.

Render detachment is the loss of adhesion between the mortar and the support masonry (Figure 2.20a). Possible causes for this defect are (Magalhães 2002; Rodrigues 2006): errors in mortar production and application; prolonged presence of excessive moisture on the wall (as pointed out previously); cryptoflorescences; deformation due to variations in temperature or moisture content; and support movements. In the facade walls studied, detachment has frequently led to the loss of the render layer (Figure 2.20a). In these cases, the adobes have become exposed to the direct action of weathering agents.

In some of the buildings where cement render was used, the render has cracking and detachment problems and the support adobes degradation problems. These defects are a result of the incompatibility between the properties of cement mortars and adobe masonry (May 2005; Rodrigues 2006). Cement mortar has greater stiffness than adobe, and the two materials have very different moisture and thermal expansion rates, factors that frequently lead to the cracking of mortar and separation from the support (USDOI 1978; Rodrigues 2006). In addition, cement mortar hinders exchanges of water vapour, causing the retention of this vapour and the concentration of salts in the support structure, which then contribute to accelerate the degradation of the support (Rodrigues 2006).

Render erosion is the destruction or wear of the render (Figure 2.20b) with loss of material or only the alteration of the render surface (Magalhães 2002). It is caused by the direct action of weathering agents (particularly rain, temperature variations, and wind) or other mechanical agents that induce stresses in the material (Magalhães 2002). In a few of the walls observed where erosion has been particularly intense, the adobes have become exposed. In one of the buildings studied (‘*H*21’), the unprotected adobes are highly degraded.

2.5.4.2. Paint deterioration

Paint deterioration was observed in 67% of the buildings studied. Two types of deterioration were identified: peeling and erosion. The peeling effect was mainly observed in the recent layers of impermeable paint (Figure 2.20c), which are not compatible with the support structure and create a barrier to the passage of moisture (Rodrigues 2006; Tavares et al. 2012). The erosion effect was observed only in lime paint. In these cases, the paint has been partially eroded due to the action of weathering agents, and the support lime render has become visible (Figure 2.20d).

2.5.4.3. Ceramic tile deterioration

In half of the buildings that have facade walls with ceramic tiles, the tiles show mild superficial deterioration (Figure 2.20e). In one building (‘*H*35’), there are small wall areas with missing tiles (Figure 2.20e). The main cause of tile deterioration is the action of weathering agents or other mechanical agents that induce stresses in the material.

2.5.4.4. Final comments

The main causes of deterioration of the external finishes of the facade walls studied are the action of weathering agents (especially rain, temperature variations, and wind) and the incompatibility between the selected finishing solution (namely, cement mortar or paint with impermeable characteristics) and the support structure. The external finish of facade walls is critical to protect these elements from the action of weathering and other degradation agents. The deterioration of this protective layer leads to the exposure of the support structure, which then becomes more vulnerable to degradation. The protection, regular maintenance, and repair of wall finishes and the use of compatible finishing solutions are thus fundamental.

2.6. State of conservation of facade walls

The state of conservation of the facade walls of the buildings studied was assessed using a rating scale that varies between 1 and 5. For each building, one rating was assigned to the masonry structure and another to the exterior finish of the facade walls. Each rating corresponds to a global evaluation of the facade walls of the building, taking into account the state of conservation of each wall. In four buildings of Aveiro municipality, it was not possible to observe the exterior finish of all the walls. In these cases, the rating assigned refers to the wall finish that could be observed.

The state of conservation of the masonry structure of each facade wall was evaluated as follows:

- 1 (‘very poor’): the wall has severe defects that greatly affect its structural integrity; it may have suffered partial or complete collapse;
- 2 (‘poor’): the wall has defects that affect its structural integrity;
- 3 (‘reasonable’): the wall has defects that do not affect – or only slightly affect – its structural integrity; relatively simple rehabilitation measures would be sufficient to resolve the existing problems;
- 4 (‘good’): the wall has few defects; the existing defects are of low to moderate intensity;
- 5 (‘very good’): the wall does not have significant defects.

The state of conservation of the exterior finish of each facade wall was evaluated as follows:

- 1 (‘very poor’): the wall finish has severe defects that significantly compromise its performance;
- 2 (‘poor’): the wall finish has defects that compromise its performance;
- 3 (‘reasonable’): the wall finish has defects that do not affect – or only slightly affect – its performance; relatively simple rehabilitation measures would be sufficient to resolve the existing problems;
- 4 (‘good’): the wall finish has few defects; the existing defects are of low to moderate intensity;
- 5 (‘very good’): the wall finish does not have significant defects.

In the evaluation of the state of conservation of the facade walls, where there was uncertainty between two consecutive ratings, the scale was refined using the mean of the two ratings (i.e. 1.5, 2.5, 3.5 or 4.5).

This rating system was used to enable a quick assessment of the state of conservation of the facade walls. With this simple system, it is possible to acquire a general knowledge of the state of conservation of a significant number of buildings. For a more accurate evaluation and understanding of the state of conservation of a specific building, however, a more in-depth analysis is recommended.

The number and percentage of buildings per rating of state of conservation, for the masonry structure and exterior finish of the facade walls, for each municipality, are presented in Figure 2.21. The same information, considering the three municipalities in conjunction, is also presented.

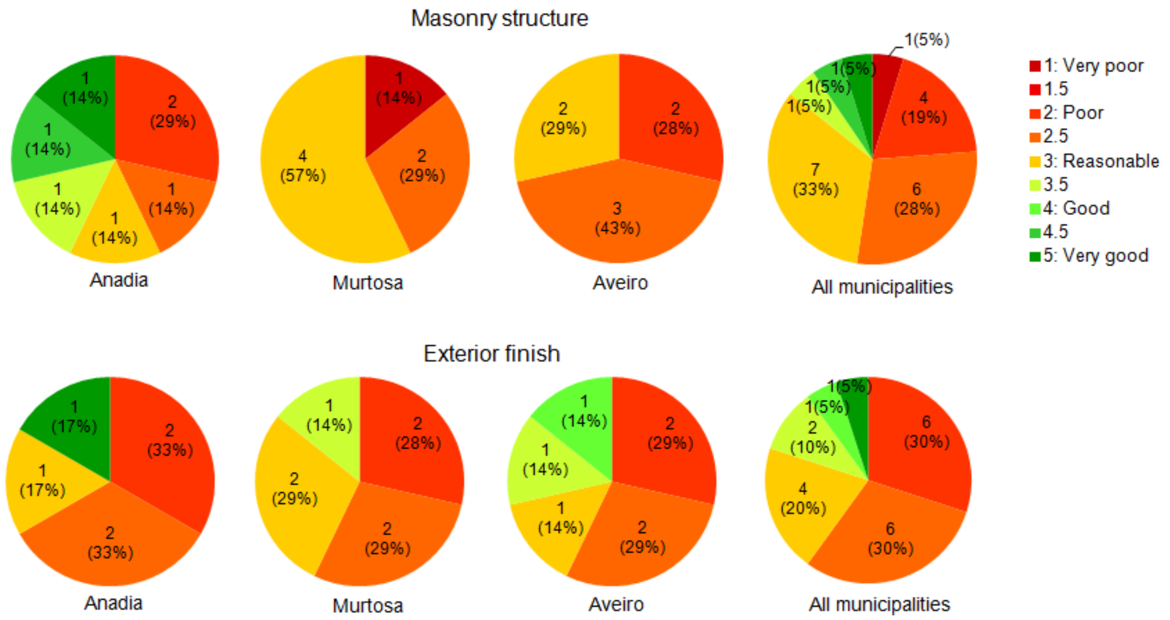


Figure 2.21: State of conservation of facade walls.

Considering all the municipalities studied, a wide interval of state of conservation ratings is observed. These ratings vary between 1 and 5, for the masonry structure, and between 2 and 5, for the exterior finish. The facade walls of the buildings analysed in Anadia municipality have the highest ratings of state of conservation. This is mainly due to the fact that the three buildings of most recent construction (built after 1950), two of which (‘*H*23’ and ‘*H*25’) have been subject to regular rehabilitation interventions, are located in this municipality. In general, mainly due to this fact, there is a slightly greater concentration of defects in the buildings studied in the other municipalities (Murtosa and Aveiro).

Frequently, the state of conservation of the masonry structure is close to or higher than that of the exterior finish. In the buildings for which the state of conservation of the masonry structure is significantly lower than that of the exterior finish (‘*H*34’, ‘*H*39’, and ‘*H*40’), this difference is due to the fact that some of the facade walls of the buildings display severe structural cracking or partial collapse, while the wall finish, overall, is in a reasonable state of conservation.

The masonry structure of the facade walls of 52% of the buildings studied is in a state of conservation rated below ‘reasonable’. This percentage rises to 71% when the buildings of Aveiro municipality are considered separately. The state of conservation of the exterior wall finish in 60% of the buildings was also rated below ‘reasonable’. It can thus be concluded that the facade walls of many of the adobe buildings studied are in pressing need of adequate rehabilitation measures.

2.7. Conclusions and final remarks

A visual and dimensional inspection of the facade walls of twenty-one adobe buildings selected from three municipalities of Aveiro district was carried out, and the results obtained were presented in this chapter.

The features and construction details of the facade walls studied can be summarised as follows:

- The facade walls are load-bearing, and their thickness varies approximately between 0.30 m and 0.70 m; the great majority of the buildings studied have facade walls made with ‘lime adobes’ and a few with ’mud adobes’;
- Facade wall openings are generally made with a wooden lintel positioned above the opening with its ends embedded in the adobe masonry;
- The connection between facade walls is made by the interlocking of adobe bricks in the corners;

- Five buildings have thin reinforced concrete bond beams and one building wooden bond beams (out of a total of nine buildings where observation was possible);
- In general, the limits indicated in technical standards for the distance between lateral supports and slenderness ratios of facade walls are not respected;
- Five buildings have stone masonry foundations and two buildings adobe masonry foundations (observation was not possible in other buildings);
- The facade walls of most buildings are rendered with lime mortar; in some cases, a more recent layer of cement mortar was applied over the existing layer of lime mortar;
- In a large percentage of buildings, the facade walls were finished with lime paint; in some buildings, other types of paint, generally with impermeable characteristics, were added later; ceramic tiles are also found in a significant percentage of buildings.

The following key conclusions regarding the defects and state of conservation of the facade walls can also be drawn:

- The main types of defects observed in the facade walls are cracking, dampness, and wall finish deterioration;
- The main possible causes of structural cracking identified are the deformation of walls due to variations in temperature or moisture content, differential foundation settlement, and excessive load imposed by the roof or floor structures;
- The major causes of the dampness problems observed are the lack or malfunction of the rainwater drainage systems and the existence of deficiencies in the roof and window openings that lead to rainwater penetration;
- The main causes of deterioration of the exterior finish of the facade walls are the action of weathering agents and the incompatibility between the selected finishing solution and the support structure;
- Many of the buildings studied have facade walls in a state of conservation rated below ‘reasonable’.

It can be concluded that the facade walls of a large percentage of buildings have defects and vulnerabilities that compromise their performance and, in some cases, the structural integrity of the buildings. The rehabilitation and strengthening of these key structural elements, addressing and correcting the causes of existing defects, are fundamental. The execution of regular maintenance work is also essential for their good performance. It is important to note that, despite the existing defects and vulnerabilities, many adobe buildings, including those that are currently vacant, if adequately rehabilitated and strengthened, can perform their function well.

The work developed and presented in this chapter aims to contribute to a better understanding of the building systems and defects of existing adobe buildings. The enrichment of this knowledge is essential to support the conservation and rehabilitation of the existing adobe built heritage, not only in Portugal but also in other regions of the world. It is important to note, however, that a greater number of adobe buildings per municipality must be studied, in order for the results to be statistically representative. This first study is important since it allowed to test and improve the survey strategy adopted, and the results obtained are relevant because this type of information did not exist for the areas under study. However, the results obtained are preliminary and must be expanded in future work.

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Chapter 3

Compressive and tensile strength of adobe bricks

The work reported in this chapter is presented in: Silveira, D., Varum, H., Costa, A., Martins, T., Pereira, H., and Almeida, J. (2012). “Mechanical properties of adobe bricks in ancient constructions.” *Constr. Build. Mater.,* 28(1), 36-44.

3.1. Introduction

In the investigation of the behaviour of adobe masonry, the study of the mechanical properties and behaviour of the constituent materials (adobes and mortars) is an important first step (Morel et al. 2007). Taking this into consideration, and given the lack of existing knowledge regarding the adobes traditionally used in Aveiro district, in Portugal, a study of the mechanical properties of adobe bricks collected from houses and land dividing walls in this region, representative of the existing adobe construction, was conducted and is presented in this chapter.

In this study, cylindrical adobe specimens were subjected to simple compression and splitting tests (also known as diametral compression tests). With these tests it was possible to evaluate the strength of the material, in compression and tension. The correlation between the tensile strength and compressive strength of the specimens was studied, and the results determined for houses and land dividing walls were compared. Comparisons of the strength values obtained with the strength limits indicated by different earthen construction technical standards and with the strength values obtained by other authors for adobes representative of adobe construction in different countries were also carried out. The results obtained are important for the characterisation of traditional adobes and can be used to support the study of adobe masonry (including, for example, in the calibration of numerical models) and rehabilitation and strengthening interventions carried out on adobe constructions.

3.2. Technical standards and recommendations

A research of the existing technical standards and recommendations for adobe construction was conducted. The following documents, which were considered the most complete, were carefully analysed: ‘NZS 4297:1998 Engineering design of earth buildings’ (SNZ 1998a); ‘NZS 4298:1998 Materials and workmanship for earth buildings’ (SNZ 1998b); ‘NZS 4299:1998 Earth buildings not requiring specific design’ (SNZ 1998c); ‘Norma técnica de edificación NTE E.080 Adobe’ (ICG 2006); ‘2009 New Mexico earthen building materials code’ (RLD 2009); ‘The Australian earth building handbook’ (Walker 2002).

The documents analysed have guidelines for the testing of materials for new constructions, but the materials that are analysed within this study were collected from old constructions, some of which are in a poor state of conservation. Considering this and the limitations of the available laboratory facilities, it was concluded that in the execution of the tests it would not be possible to strictly comply with normative recommendations. The indications of the Australian handbook (Walker 2002), which are the most adequate considering the available laboratory facilities, were used as references but not as strict rules.

The documents consulted indicate flexural tests for the determination of the tensile strength of adobe. However, it was decided to conduct splitting tests, instead, as these are more adequate to the existing laboratory facilities. In addition, splitting tests present some advantages when compared to flexural tests – a splitting test more closely resembles a direct tension test, and the results obtained are less variable than in a flexural test (according to studies on the testing of concrete specimens) (Ozyildirim and Carino 2006).

The RILEM technical recommendation ‘CPC 6 Tension by splitting of concrete specimens’ (RILEM 1994), which is addressed to concrete, was used as reference in the execution of the splitting tests.

3.3. Selection, preparation, and testing of specimens

3.3.1. Adobes

For the experimental testing campaign, a set of lime stabilised adobes, representative of different adobe construction typologies, were selected from eight houses and eight land dividing walls, from different locations in Aveiro district. The adobe bricks were in a good state of conservation and their mean dimensions are: 0.45 x 0.30 x 0.12 m^3, for houses; and 0.45 x 0.20 x 0.12 m^3, for land dividing walls. The mean specific weight of the adobes is 16 kN/m^3 (CV = 5%).

3.3.2. Specimens

The technical standards and recommendations for earthen construction analysed in this study (SNZ 1998b; ICG 2006; RLD 2009; Walker 2002) indicate that simple compression tests shall be conducted on adobe bricks or cubic specimens. These documents address new constructions, and thus the test specimens considered can be specifically moulded for testing. The Australian handbook (Walker 2002) indicates the possibility of testing cylindrical specimens.

In this study, tests were conducted on cylindrical specimens, for the following reasons:

i) Considering the limitations of the available laboratory facilities, the extraction of cylindrical specimens from adobe bricks is simpler than the extraction of cubic specimens as it only implies cutting and regularizing three surfaces; it is important to note, however, that the extraction of cylindrical specimens is only viable for adobe units that have good cohesion of aggregates and do not have excessively large particles in their composition – as was the case of the majority of the adobes collected –, because these particles can damage specimens during the extraction process;

ii) When simple compression tests are conducted on cylinders with a height to diameter ratio of 2, the failure stress is closer to the unconfined compressive strength, when compared to that obtained by testing cubes, because the effects of end restraint are reduced (according to studies on the testing of concrete specimens) (Illston and Domone 2001);

iii) According to the 'CPC 6' RILEM technical recommendation (RILEM 1994), splitting tests shall be conducted on cylindrical specimens with a height to diameter ratio of 2, and thus the process of preparation of specimens for both tests was simplified.

Cylindrical specimens were extracted by rotary core drilling from the adobe bricks collected, with diameters ranging from 80 to 90 mm (Figure 3.1). The variation in diameters is due to the different levels of erosion suffered by the specimens during the cutting process. A few specimens were extracted with smaller diameters due to defects in the adobe bricks. The cylinders were cut with a height to diameter ratio of approximately 2, whenever possible, and never less than 1 (RILEM 1994). For the specimens with a height to diameter ratio equal to or less than 1.75, tested in simple compression, correction factors were applied in the calculation of compressive strength. Given that there are no specific correction factors for adobe specimens, factors for concrete were used (ASTM 2012).

Figure 3.1: Cylindrical cores extracted from the adobe bricks.

To facilitate the identification of the specimens and the analysis, the adobe cylindrical specimens were labelled according to their origin and to the type of test conducted. The notation $\begin{Bmatrix} H \\ W \end{Bmatrix} i\_a\_ \begin{Bmatrix} clc \\ clt \end{Bmatrix} k$ was adopted, distinguishing:

- Adobe specimens ('a') from houses ('H') and land dividing walls ('W');

- Cylindrical adobe specimens subjected to simple compression (*'clc'*) and splitting (*'clt'*) tests.

Index *'i'* represents the number of the construction from which the adobe was collected and index *'k'* the number of the cylindrical specimen.

3.3.3. Testing

A total of 101 cylindrical specimens, 51 collected from houses and 50 from land dividing walls, were subjected to mechanical tests, using a universal mechanical compression testing machine (Shimadzu Autograph AG 25 TA). 83 specimens were submitted to compression and 18 to splitting tests (Figure 3.2).

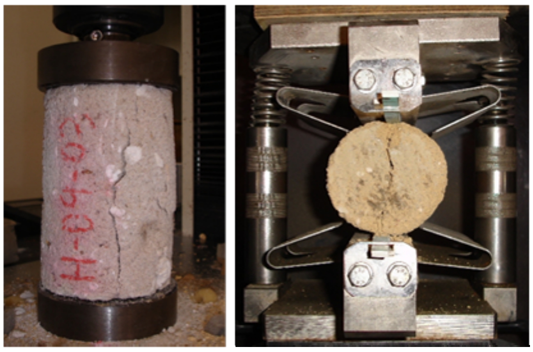

Figure 3.2: Simple compression and splitting tests on adobe specimens.

In the tests performed, a uniform load was applied without shock and increased continuously until failure, with the moving head of the testing machine travelling at a rate of 1 to 2 mm/min, respecting the testing rate interval (1 to 5 mm/min) recommended by the Australian handbook (Walker 2002).

The testing rate limits indicated in the 'CPC 6' RILEM technical recommendation (RILEM 1994), for splitting tests, are for load-controlled devices, and the available testing machine is strain-controlled. In addition, concrete is a material with higher strength and stiffness than adobe, and thus the use of these rate limits would be inadequate for the testing of adobe specimens. Therefore, in the conduction of splitting tests a rate of 1 to 2 mm/min was also adopted.

3.4. Results

3.4.1. Compressive strength

The compressive strength of the adobe bricks was obtained by testing the cylindrical specimens in simple compression (Table 3.1). The compressive strength (‘f_c’) is given by $f_c = F_c / A$, where ‘F_c’ is the failure load, and ‘A’ is the cross-sectional area that resists the load.

Table 3.1: Results obtained in the mechanical tests conducted on adobe specimens.

Construction		Compressive strength		Tensile strength	
		Mean (MPa)	CV (%)	Mean (MPa)	CV (%)
Houses	*H*01	1.24	9	0.13	49
	*H*02	1.00	23	0.19	11
	*H*03	0.75	20	0.19	23
	*H*04	0.66	25	...	...
	*H*05	2.15	41	...	...
	*H*09	0.70	30	...	...
	*H*10	1.98	29	...	...
	*H*11	1.08	22	...	...
Land dividing walls	*W*01	0.94	26	...	...
	*W*02	0.83	32	0.13	65
	*W*04	0.99	21	0.12	24
	*W*05	1.72	17	0.40	31
	*W*06	1.25	23	...	...
	*W*07	0.80	25	...	...
	*W*09	1.05	50	...	...
	*W*10	0.98	18	...	...

The mean compressive strength, calculated per construction under analysis, ranges between 0.66 MPa (‘*H*04’) and 2.15 MPa (‘*H*05’) (Table 3.1 and Figure 3.3). The global mean compressive strength for specimens taken from houses is 1.32 MPa and for specimens from land dividing walls is 78% of that value (1.03 MPa) (Figure 3.3). The results obtained for some of the constructions analysed have high variability, as expressed by the standard deviations presented in Figure 3.3. The variability considering all the results obtained for different constructions is also significant and is larger for houses, which have the lowest and highest mean compressive strength values (Figure 3.3).

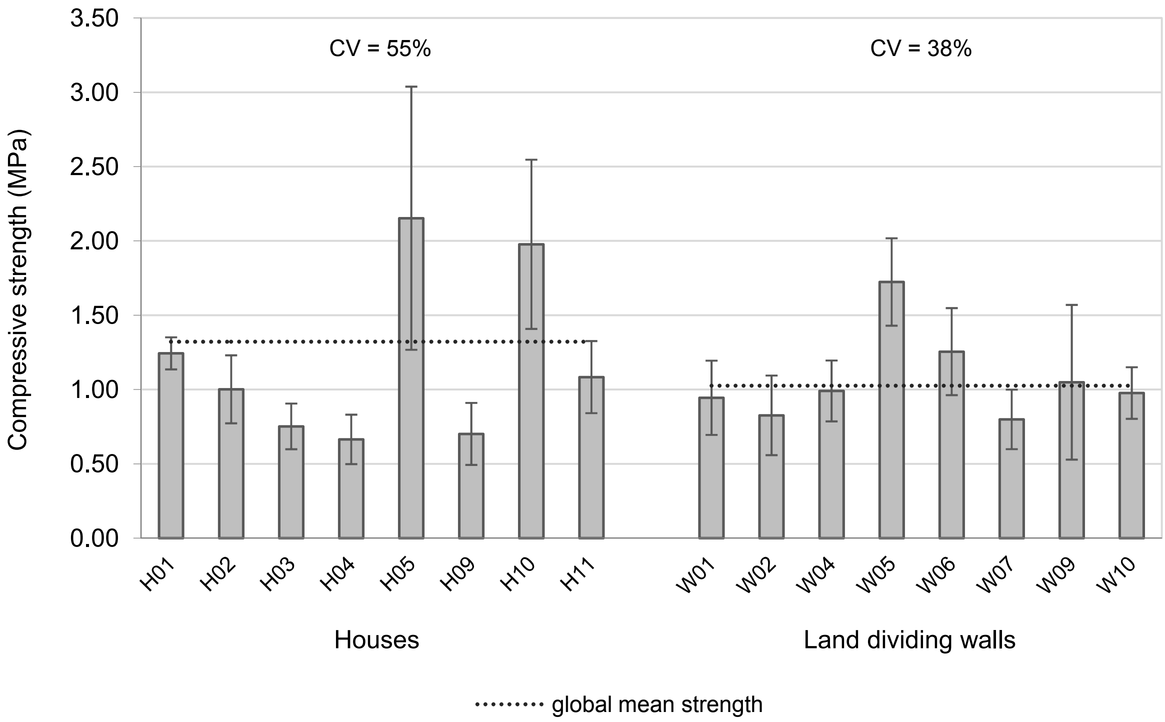

Figure 3.3: Mean compressive strength of adobe specimens, per construction under study, with indication of standard deviation.

3.4.2. Tensile strength

The tensile strength of the adobe bricks was obtained from splitting tests performed on cylindrical specimens (Table 3.1). The tensile strength (‘f_t’) is given by $f_t = \dfrac{2F_t}{\pi DH}$ where ‘F_t’ is the failure load, ‘D’ is the diameter of the specimen, and ‘H’ is the height of the specimen.

The mean tensile strength, calculated per construction under study, varies between 0.12 MPa (‘*W*04’) and 0.40 MPa (‘*W*05’) (Table 3.1 and Figure 3.4). The global mean tensile strength for specimens collected from land dividing walls is 0.22 MPa and for specimens from houses is 78% of that value (0.17 MPa) (Figure 3.4). The results obtained for some of the constructions studied have high variability, as expressed by the standard deviations displayed in Figure 3.4. The variability considering all the results obtained for different constructions is also considerable and, contrary to what was observed in the study of compressive strength, is greater for land dividing walls (Figure 3.4).

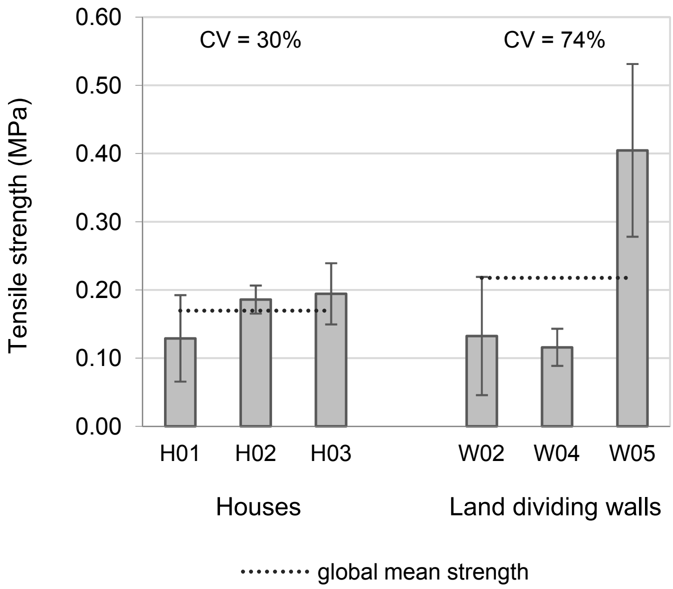

Figure 3.4: Mean tensile strength of adobe specimens, per construction under study, with indication of standard deviation.

3.4.3. Correlation between tensile strength and compressive strength

The correlation between the tensile strength and compressive strength of the specimens tested was studied. For each construction analysed, the mean tensile strength was plotted against the respective mean compressive strength, and the best-fit linear correlation was determined (Figure 3.5). According to this best-fit correlation, tensile strength corresponds to approximately 18% of compressive strength.

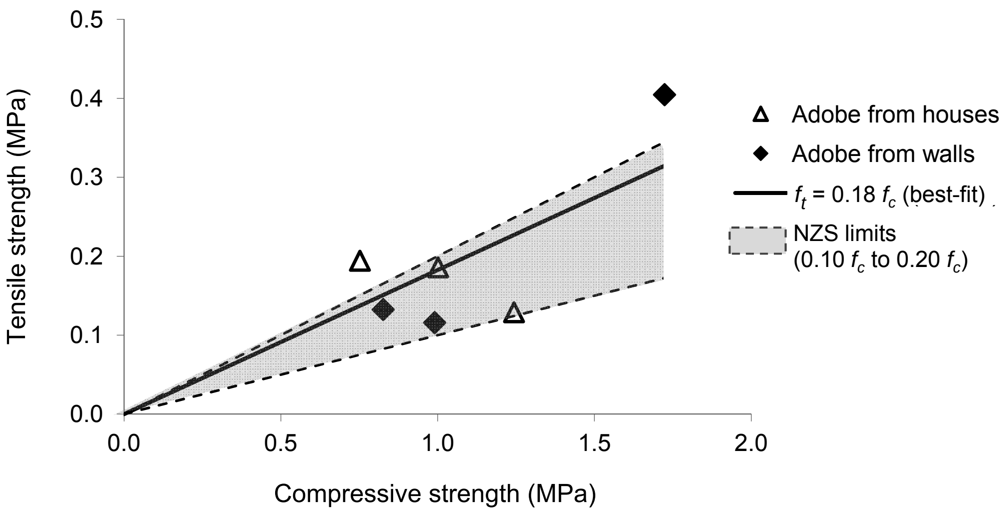

Figure 3.5: Correlation between tensile strength and compressive strength, with indication of the limits presented in 'NZS 4298' (SNZ 1998b).

'NZS 4298' (SNZ 1998b) indicates that flexural tensile strength generally lies between 10% and 20% of compressive strength and that the majority of results are below 30%. Thus, the correlation obtained for the adobe specimens under study, considering the tensile strength obtained from splitting tests, is within the limits suggested in this standard for flexural tensile strength (Figure 3.5).

3.4.4. Comparison with normative limits

Tables 3.2 and 3.3 present the limits indicated by different technical standards for compressive and tensile strength and the number of constructions analysed that respect these limits.

Table 3.2: Evaluation of the compressive strength values obtained by comparison with normative limits.

Standard	Compressive strength limit	No. of constructions analysed that respect the limit	
		Houses (out of a total of 8)	Walls (out of a total of 8)
'NZS 4298' (New Zealand) (SNZ 1998b)	• Least of the individual results in the set > 0.7 x 1.30 MPa [a].	3	1
'NTE E.080' (Peru) (ICG 2006)	• Compressive strength value that is exceeded in 80% of the specimens tested [b] ≥ 0.7 x 1.18 MPa.	3	3
'14.7.4 NMAC' (New Mexico) (RLD 2009)	• Mean compressive strength ≥ 2.07 MPa [c]. • One sample out of the total may have a compressive strength of not less than 1.72 MPa [c].	0	0

[a] This standard indicates cubic specimens for the compressive test. In the calculation of the unconfined compressive strength limit, an aspect ratio factor of '0.7', which is indicated in the standard for cubic specimens, was introduced. This strength limit is for 'standard grade earth construction', as defined in the standard.

[b] The compressive strength value that is exceeded in 80% of the specimens tested was calculated considering a normal distribution of results.

[c] This standard indicates that the compressive test shall be conducted on adobe blocks, in the flat position, but it does not indicate the dimensions of the blocks to be tested nor recommends the use of an aspect ratio factor to take the confinement effect into account; therefore, these limit values are here considered without any correction and thus are significantly larger than the other limit values presented.

Table 3.3: Evaluation of the tensile strength values obtained by comparison with normative limits.

Standard	Tensile strength limit [a]	No. of constructions analysed that respect the limit	
		Houses (out of a total of 3)	Walls (out of a total of 3)
'NZS 4298' (New Zealand) (SNZ 1998b)	• Least of the individual results in the set > 0.25 MPa [b].	0	1
'NTE E.080' (Peru) (ICG 2006)	• No indication.	...	...
'14.7.4 NMAC' (New Mexico) (RLD 2009)	• Mean tensile strength ≥ 0.34 MPa.	0	1

[a] Flexural tensile strength.
[b] This strength limit is for 'standard grade earth construction', as defined in 'NZS 4298'.

The strength values obtained for the adobe specimens tested are, in general, lower than the established limits. This comparative analysis is not rigorous, given that the strict indications of each standard for the execution of tests were not followed and there are differences between splitting tensile strength and flexural tensile strength (compared in Table 3.3) that result from the different characteristics of the testing procedures and specimens used. This analysis is only intended to provide a general indication of the quality of the adobes studied, in terms of mechanical strength, when compared to what is required for new constructions.

3.4.5. Comparison with the results obtained by other authors

Strength values obtained by other authors (Gavrilovic et al. 1998; Meli 2005; Rivera and Muñoz 2005; Liberatore et al. 2006; Baglioni et al. 2010) for adobes representative of adobe construction in different countries are presented in Table 3.4, together with the mean results obtained in the present work. In two of the studies by other authors, it is indicated that simple compression tests were conducted on adobe bricks or half bricks, but the authors do not indicate whether a correction was made to account for the confinement effect. In the others studies, it is not indicated the type of test specimen used. The tensile strength values presented in these studies result from three point flexural tests, while the values obtained in the present work result from splitting tests. Therefore, as in the previous subsection, this comparative analysis is not rigorous and aims only to give a general indication of the quality of the adobes analysed, by comparison with adobes used in other regions of the world.

Table 3.4: Strength of adobe specimens obtained by different authors.

Reference	Location	Adobe bricks		Compression		Tension	
		Composition	Condition	Specimens	Comp. strength (MPa)	Test	Tensile strength (MPa)
Present study	Portugal	Arenaceous soil and lime binder	Collected from existing constructions	Cylinders	1.17	Splitting	0.19
Gavrilovic et al. (1998)	Mexico	Clayey soil	Not indicated	Not indicated	1.18	Flexural	0.27
Meli (2005)	Mexico	Clayey soil	New (produced in different regions of the country)	Not indicated	0.51-1.57	Flexural	0.20-0.43
Rivera & Muñoz (2005)	Colombia	Clayey soil	Collected from existing construction	Bricks	3.04	Flexural	0.41
Liberatore et al. (2006)	Italy	Silty sand	Collected from existing constructions	Bricks and half bricks	0.29-1.56	Flexural	0.17-0.40
Baglioni et al. (2010)	Morocco	Silty or clayey soil	Some collected from existing constructions and some new	... [a]	2.83	Flexural	0.18-0.35

[a] Results obtained from in situ sclerometer tests.

It can be observed that the values obtained in this work are within the range of results obtained by other authors. Higher compressive strength values, in some of the other studies, may be justified, in part, by the confinement effect in the testing of adobe blocks and, in one study, by the use of the in situ sclerometer test to evaluate the compressive strength. The tensile strength values obtained in the present work are near the lower values obtained by other authors, which may also be explained, in part, by the fact that flexural tests tend to overestimate tensile strength (Ozyildirim and Carino 2006) – as will be further explored in Chapter 4 (Silveira et al. 2013).

3.5. Conclusions and final remarks

Simple compression and splitting tests were carried out on adobe specimens extracted from adobes taken from existing constructions in Aveiro district, and the results obtained were presented and analysed in this chapter.

A summary of the mean results obtained is displayed in Table 3.5. Adobe specimens from houses have mean compressive strength values greater than those from land dividing walls, and the opposite is observed for tensile strength. The strength values determined are, in general, lower than the limits indicated in standards for earthen construction (SNZ 1998b; ICG 2006; RLD 2009) but are within the interval of values obtained by other authors for adobes representative of adobe construction in other countries (e.g. Meli (2005), Liberatore et al. (2006)).

Table 3.5: Mean results obtained in the tests performed.

	Mean compressive strength (MPa)	Mean tensile strength (MPa)
Houses	1.32	0.17
Land dividing walls	1.03	0.22
All constructions	1.17	0.19

The results obtained for some of the houses and land dividing walls studied have high variability. The variability considering all the results obtained for different constructions is also significant. High variability of results was expected since, traditionally, the materials used in the production of adobes had important heterogeneities and there were variances in production and curing procedures, even within the same construction process. Other authors also report high variability of results when testing adobe specimens representative of the adobes used in other regions of the world (e.g. Meli (2005), Liberatore et al. (2006)).

A lack of comprehensive European and international standards devoted to earthen construction was observed in the development of this study. The inexistence of recommendations for earthen construction in the Eurocodes was also verified. Furthermore, the existing standards are not complete and require improvements. These documents focus on the construction of new buildings and should also consider the rehabilitation of existing constructions, since there is a significant earthen built heritage in need of adequate repair and strengthening interventions. The development of technical standards focused on earthen construction, addressing not only new building processes but also the conservation and rehabilitation of existing constructions, is thus fundamental.

The results obtained in this study are important for the characterisation of the adobes used in traditional masonries. These results can be considered as references in further studies of adobe masonry and in the rehabilitation and strengthening of existing constructions. It should be noted, however, that tests performed on adobe specimens can only be used as indicators of the quality of adobe and not of masonry (Ottazzi 1998; ICG 2006). Thus, studies focused on the characterisation of the mechanical behaviour of the adobe masonry system are also essential.

3.6. References

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Chapter 4

Stress-strain relationships and influence of the testing procedures in the mechanical characterisation of adobe bricks

The work reported in this chapter is presented in: Silveira, D., Varum, H., and Costa, A. (2013). "Influence of the testing procedures in the mechanical characterization of adobe bricks." *Constr. Build. Mater.,* 40, 719-728.

4.1. Introduction

Technical standards that address adobe construction (e.g. SNZ (1998a), ICG (2006), RLD (2009)) indicate that simple compression tests shall be performed on adobe bricks or cubic specimens. 'The Australian earth building handbook' (Walker 2002) allows the possibility of testing adobe bricks or cylindrical specimens. In addition, most of these documents recommend the conduction of flexural tests on adobe bricks.

In the first experimental campaign, presented in Chapter 3, simple compression tests and splitting tests were conducted on cylindrical adobe specimens (Silveira et al. 2012). As explained in Chapter 3, given the characteristics of the available laboratory facilities, the extraction of cylindrical specimens from adobe bricks is easier than the extraction of cubic specimens – this is true if the material does not have excessively large particles in its composition and presents good cohesion of aggregates. Additionally, the testing of cylindrical specimens has other important advantages. In simple compression tests conducted on cylinders with a height to diameter ratio of approximately 2, the stress distribution is closer to uniaxial and the strength obtained is closer to the unconfined compressive strength, when compared to simple compression tests performed on cubic specimens (according to studies conducted on concrete specimens) (Domone 2001). Conducting simple compression tests on cylindrical specimens also allows an easier and more accurate measurement of the deformation of specimens. Cylindrical specimens have greater height than cubic specimens, which facilitates the mounting of displacement transducers. According to studies conducted on concrete specimens, splitting tests also have advantages when compared to flexural tests – a splitting test is closer to a direct tensile test, and the results obtained are less variable than those obtained in flexural tests (Ozyildirim and Carino 2006).

Several authors have conducted studies for the mechanical characterisation of adobes taken from existing constructions in different parts of the world (e.g. Rivera and Muñoz (2005), Liberatore et al. (2006), Baglioni et al. (2010), Fratini et al. (2011)) and also of adobes produced in the laboratory, usually to reproduce existing damaged adobes or to study the effectiveness of different possible compositions (e.g. Quagliarini and Lenci (2010), Fratini et al. (2011), Eslami et al. (2012)). Generally, compression tests are conducted on adobe bricks or adobe cubic specimens (e.g. Rivera and Muñoz (2005), Liberatore et al. (2006), Baglioni et al. (2010), Quagliarini and Lenci (2010), Fratini et al. (2011), Eslami et al. (2012)), without reference to the influence of confinement effect on the obtained results, and normally tensile strength of adobe is evaluated by conducting flexural tests (e.g. Rivera and Muñoz (2005), Liberatore et al. (2006)). In general, the existing knowledge regarding the influence of the geometry of specimens and testing procedures on the mechanical characterisation of earthen specimens is very limited. Nevertheless, the importance of such study has been recognised and the first steps to contribute to this knowledge have been taken (e.g. Morel et al. (2007)).

It would be useful to have the possibility to test cylinders or cubes and to conduct flexural or splitting tests, depending on the characteristics of adobes and existing conditions for the extraction and testing of specimens. For adobe, however, there are no studies establishing correlations between the results obtained with these different procedures.

In view of this, and with the aim of contributing with an initial proposal for the correlations between mechanical properties determined with different testing procedures, a series of experimental tests were carried out and the results obtained are presented in this chapter. Cylindrical and cubic adobe specimens were subjected to simple compression tests, adobe bricks to three point flexural tests, and cylindrical adobe specimens to splitting tests. The test specimens were extracted from adobe bricks collected from representative houses in Aveiro district, in Portugal. In addition to contribute to the understanding of the influence of the experimental testing procedures used in the mechanical characterisation of adobe, with this work it was also possible to gather more data to improve the knowledge about the mechanical properties and behaviour of the material – including data resulting from the study of the deformations suffered by specimens under compression. In the previous experimental campaign, presented in Chapter 3, due to existing laboratory limitations, it was not possible to measure the deformation of specimens during testing. In the present campaign, however, it was possible to use displacement transducers mounted directly on the test specimens, which allowed the study of the stress-strain curves, modulus of elasticity, and Poisson's ratio of the material. Three theoretical stress-strain curves, calibrated with the results obtained, were proposed, and the correlation between modulus of elasticity and compressive strength was assessed. The knowledge gained is relevant to assist the study of the behaviour of adobe masonry (such as in the calibration of numerical models) and, in general, to support rehabilitation and strengthening interventions on existing adobe buildings.

4.2. Selection, preparation, and testing of specimens

4.2.1. Adobes

A total of 31 lime stabilised adobes were collected from three houses, in different locations of Aveiro district (Figure 4.1): i) 11 adobes from house *'H12'* (undergoing reconstruction), located in the parish of Bunheiro, in Murtosa municipality; ii) 10 adobes from house *'H*13*'* (undergoing demolition), located in the parish of Monte, in Murtosa municipality; and iii) 10 adobes from house *'H*20*'* (undergoing demolition), located in the parish of Cacia, in Aveiro municipality.

Figure 4.1: Adobe houses: a) *'H*12*'*; b) *'H*13*'*; c) *'H*20*'* (Costa et al. 2007).

The adobe bricks collected were protected by lime render in the buildings and were in a good state of conservation (Figure 4.2). The mean dimensions and specific weight of the adobes collected are, respectively: 0.41 x 0.28 x 0.13 m^3 and 16 kN/m^3 (CV = 6%), for *'H*12*'*; 0.46 x 0.32 x 0.12 m^3 and 15 kN/m^3 (CV = 5%), for *'H*13*'*; 0.44 x 0.24 x 0.12 m^3 and 15 kN/m^3 (CV = 6%), for *'H*20*'*.

Figure 4.2: Adobe bricks collected from: a) '*H*12'; b) '*H*13'; c) '*H*20'.

4.2.2. Technical recommendations

The following documents were used as reference in the preparation of specimens and conduction of tests: 'The Australian earth building handbook' (Walker 2002), for simple compression and flexural tests; and the RILEM technical recommendation 'CPC 6 Tension by splitting of concrete specimens' (RILEM 1994), for splitting tests. As in the previous experimental campaign, presented in Chapter 3, the recommendations in these documents were considered as guidelines and were not strictly followed due to limitations of the available laboratory facilities and also given the fact that these documents address materials for new constructions while the present study focuses on materials collected from existing constructions. The 'CPC 6' technical recommendation for concrete (RILEM 1994) was adopted given that there is no technical recommendation for the conduction of splitting tests on cylindrical adobe specimens.

4.2.3. Specimens

Cylindrical specimens were extracted by rotary core drilling from whole adobe bricks (for simple compression tests) and from the halves of adobe bricks that resulted from flexural tests (for splitting tests), with diameters ranging from 78 to 93 mm (Figure 4.3a). This variation in diameter was caused by the erosion suffered by the material during the drilling process. As in the previous experimental campaign, presented in Chapter 3, specimens were extracted with a height to diameter ratio of approximately 2, whenever possible, and never less than 1 (RILEM 1994). For the specimens cut with a height to diameter ratio equal to or below 1.75, correction factors were used in the calculation of compressive strength. As explained in Chapter 3, there are no correction factors for adobe specimens, and thus factors for concrete were used (ASTM 2012). Cubic specimens were cut from the same adobe bricks from which cylindrical specimens (for simple compression tests) were extracted (Figure 4.3b). For flexural tests, the lateral faces of bricks were cut in order to adjust their width to the dimensions of the testing machine (Figure 4.3c). The number of specimens, per house and type of test, is indicated in Table 4.1, and the mean dimensions are presented in Table 4.2.

Figure 4.3: a) Cylindrical, b) cubic, and c) rectangular parallelepipedic test specimens.

Table 4.1: Number of test specimens.

Test	Number of test specimens		
	*H*12	*H*13	*H*20
Simple compression (cubes)	7	16	9
Simple compression (cylinders)	7 [a]	15 [b]	6 [c]
Flexural (bricks)	7	5	4
Splitting (cylinders)	10	12	7

Number of specimens where adequate measurement of deformations was possible: [a] 5; [b] 5; [c] 2.

Table 4.2: Mean dimensions of test specimens.

Specimen		Mean dimensions (m)		
		*H*12	*H*13	*H*20
Cylinder (compression and splitting tests)	*H*:	0.16	0.16	0.16
	D:	0.09	0.09	0.09
	H/*D*:	1.8	1.8	1.8
Cube (compression test)	*H* (=*L* =*W*):	0.11	0.10	0.10
Brick (flexural test)	*L*:	0.41	0.46	0.43
	W:	0.24	0.24	0.22
	H:	0.12	0.11	0.12

Notation: *H* - Height; *D* - Diameter; *L* - Length; *W* - Width.

To facilitate the identification of the test specimens and the analysis, specimens were labelled according to their origin, shape, and type of test conducted. The following notation was adopted: $Hi\_aj\_\left\{\begin{matrix} clc \\ clt \\ cb \\ pr \end{matrix}\right\}k$, where *'H'* indicates the type of construction (in this case, house), *'i'* is the index which represents the number of the construction from which the adobe brick was collected, *'a'* indicates the type of material (in this case, adobe), *'j'* is the index which represents the number of the adobe brick from which the test specimen was extracted, *'clc'* corresponds to a cylindrical specimen subjected to simple compression testing, *'clt'* corresponds to a cylindrical specimen subjected to splitting testing (tension), *'cb'* corresponds to a cubic specimen subjected to simple compression testing, *'pr'* corresponds to a rectangular parallelepipedic specimen subjected to flexural testing, and *'k'* is the index which represents the number of the test specimen.

4.2.4. Testing

Specimens were tested in the laboratory, using an ‘ELE Multiplex 50-E’ testing machine. In simple compression tests, a 50 kN load ring was used, and in flexural and splitting tests, a 10 kN load ring was used. The test setups are presented in Figure 4.4. In all simple compression tests and, when necessary, in flexural and splitting tests, a layer of fine damp sand was placed in the testing interface, for regularisation.

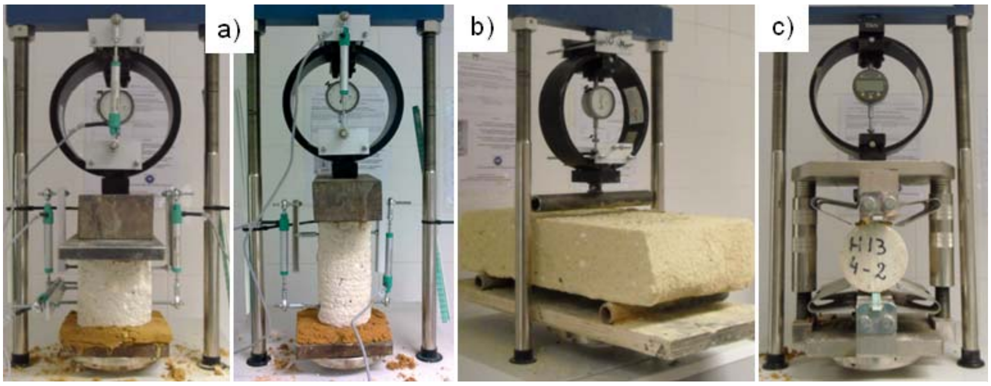

Figure 4.4: a) Simple compression, b) flexural, and c) splitting tests.

The Australian handbook (Walker 2002) recommends a testing rate of 1 to 5 mm/min for simple compression tests. The rate limits indicated in this handbook, for flexural tests, and in the ‘CPC 6’ RILEM technical recommendation (RILEM 1994), for splitting tests, are for load-controlled devices, and the available testing machine is strain-controlled; in addition, ‘CPC 6’ addresses the testing of concrete, which is a material with higher strength and stiffness than adobe. Thus, as in the previous experimental campaign, the limits indicated in the Australian handbook (Walker 2002) for simple compression tests were considered in all the tests performed. Load was applied without shock and increased continuously until failure, with the head of the testing machine moving at a rate of 1.5 mm/min.

4.2.5. Measurement of deformations

In simple compression tests, the measurement of the deformation of specimens was conducted using Gefran ‘PZ12-A’ rectilinear displacement transducers (with useful electrical stroke of 50 mm and independent linearity of 0.1%) (Figure 4.4a). In each cylindrical specimen, two or three transducers were placed in the vertical direction, equally spaced. Initially, in each cubic specimen, two transducers were placed in the vertical direction in two opposite vertical faces, and one transducer was placed in the horizontal direction in one of the vertical faces. The majority of the deformation values measured in cubes, however, were affected by different sources of error and, therefore, were excluded. The errors are possibly linked to the small dimensions of the cubic specimens, which did not allow an adequate measurement of deformations. In addition, in cubes, the stress distribution deviates from the uniaxial distribution (Domone 2001). Only the measurements conducted in one of the cubes were considered, and the Poisson's ratio was calculated for that specimen (subsection 4.3.6). The number of cylindrical specimens where adequate measurement of deformations was possible is indicated in Table 4.1.

4.3. Results

4.3.1. Compressive strength

The compressive strength ('f_c') was obtained through simple compression testing of cylindrical and cubic specimens. This strength is given by $f_c = F_c / A$, where 'F_c' is the failure load, and 'A' is the cross-sectional area that resists the load. The mean compressive strength values obtained for each adobe, for the two types of specimen, are presented in Figure 4.5, with indication of mean values and coefficients of variation per house under analysis (calculated considering the results obtained for all the individual test specimens).

The mean compressive strength, calculated per adobe under analysis, for cylindrical specimens, varies between 0.23 MPa ('*H*20_*a*07') and 1.02 MPa ('*H*12_*a*03'), with a global mean value of 0.58 MPa. For cubes, the mean compressive strength, calculated per adobe, ranges from 0.28 MPa ('*H*13_*a*10') to 1.21 MPa ('*H*12_*a*06'), with a global mean value of 0.54 MPa. The results show considerable variability, particularly the results obtained in the testing of cylindrical specimens collected from house '*H*13'. The mean compressive strength values obtained by testing cylindrical specimens are close to the lower mean values obtained in the first experimental campaign, presented in Chapter 3.

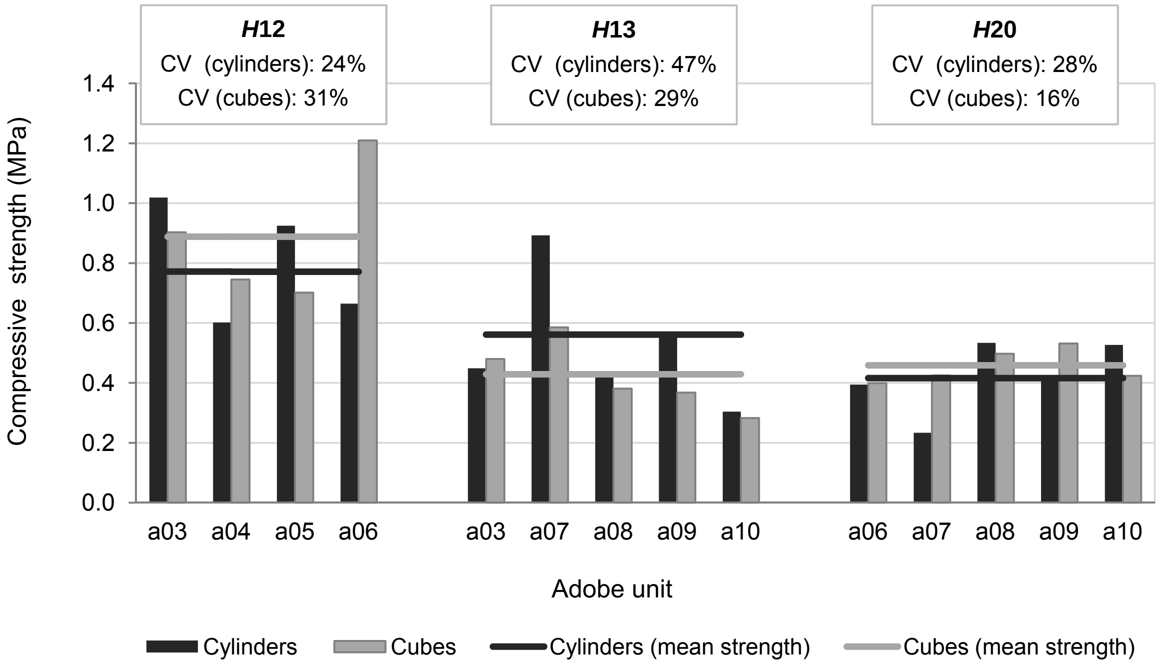

Figure 4.5: Mean compressive strength per adobe brick, obtained by testing cylindrical and cubic specimens.

4.3.2. Flexural and splitting tensile strength

The flexural tensile strength and splitting tensile strength were obtained through flexural testing of bricks and splitting testing of cylinders, respectively. The flexural tensile strength (‘$f_{t,flex}$’) is given by $f_{t,flex} = \frac{3F_t l}{2WH^2}$, where ‘$F_t$’ is the failure load, ‘$l$’ is the length of the support span, and ‘W’ and ‘H’ are the width and height of the specimen, respectively. The splitting tensile strength (‘$f_{t,split}$’) is given by $f_{t,split} = \frac{2F_t}{\pi DH}$, where ‘$F_t$’ is the failure load, and ‘D’ and ‘H’ are the diameter and height of the specimen, respectively. The mean strength values obtained for each adobe, using the two different testing procedures, are presented in Figure 4.6. The mean values and coefficients of variation, per house under study, calculated considering the results obtained for all the individual test specimens, are also indicated. In the cases for which no splitting tensile strength value is indicated, it was not possible to extract intact cylindrical specimens from the adobe halves that resulted from flexural testing.

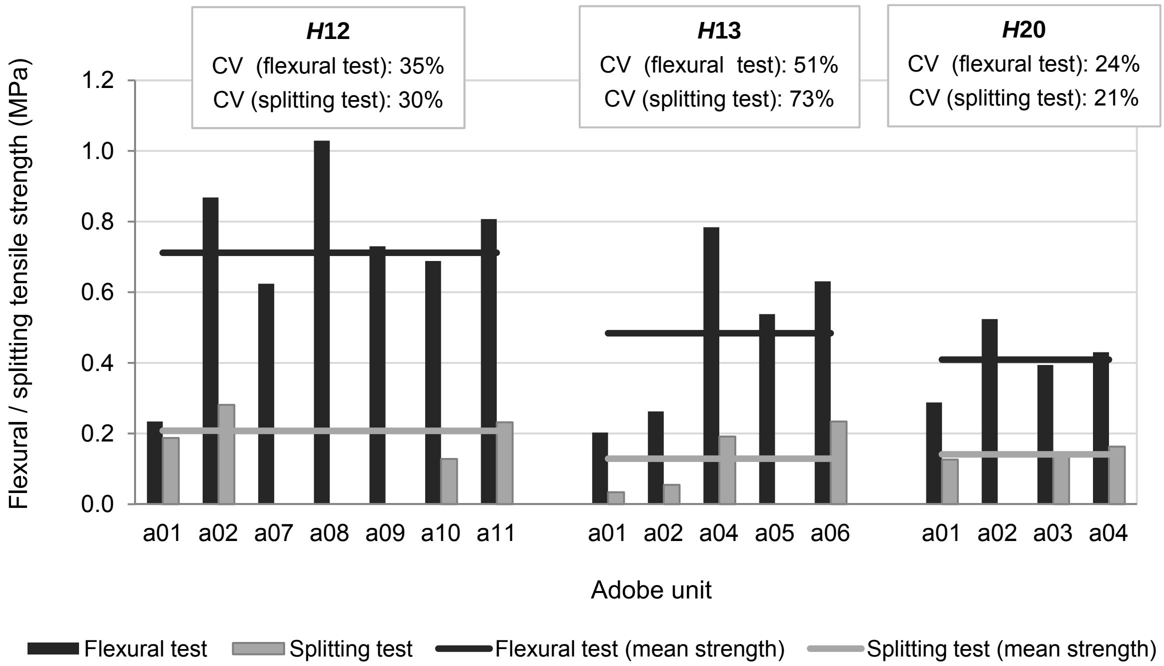

Figure 4.6: Mean flexural and splitting tensile strength, per adobe brick.

The mean flexural tensile strength, calculated per adobe, ranges between 0.20 MPa (*'H*13_*a*01*'*) and 1.03 MPa (*'H*12_*a*08*'*), with a global mean value of 0.56 MPa. The mean splitting tensile strength, calculated per adobe, varies between 0.03 MPa (*'H*13_*a*01*'*) and 0.28 MPa (*'H*12_*a*02*'*), with a global mean value of 0.16 MPa. The splitting strength values are close to the results obtained in the preceding experimental study, presented in Chapter 3.

Results show high variability, especially the results obtained for house *'H*13*'*. As previously mentioned in subsection 4.1, according to studies conducted on concrete, results from flexural tests generally have higher variability than results from splitting tests. In the present study, this was verified for houses *'H*12*'* and *'H*20*'*, but not for *'H*13*'* which, for splitting tests, has a very high coefficient of variation. More tests are necessary, comprising more buildings, to confirm if, for adobe specimens, flexural tests tend to present higher variability of results.

In the first experimental campaign, presented in Chapter 3, the compressive strength and tensile strength of adobes traditionally used in Aveiro district were compared to the strength values obtained by other authors for adobes from different parts of the world and to the strength limits indicated in earthen construction technical standards, and thus these comparisons will not be repeated here.

4.3.3. Stress-strain curves

The stress-strain curves that resulted from the simple compression tests conducted on cylindrical specimens, plotted until peak stress, are presented in Figure 4.7. In addition, three different proposals of theoretical stress-strain curves, expressed by exponential functions and calibrated with the results obtained were determined and are also presented in Figure 4.7. Knowledge of the stress-strain constitutive laws of adobe is important, because these relationships express essential information about the properties and mechanical behaviour of adobe (Gere and Goodno 2011), are useful to support the numerical modelling of the behaviour of this material, and can assist the validation of the results of experimental tests conducted by other authors. The constitutive laws proposed can be used directly in numerical modelling, or simpler laws derived from them can be used.

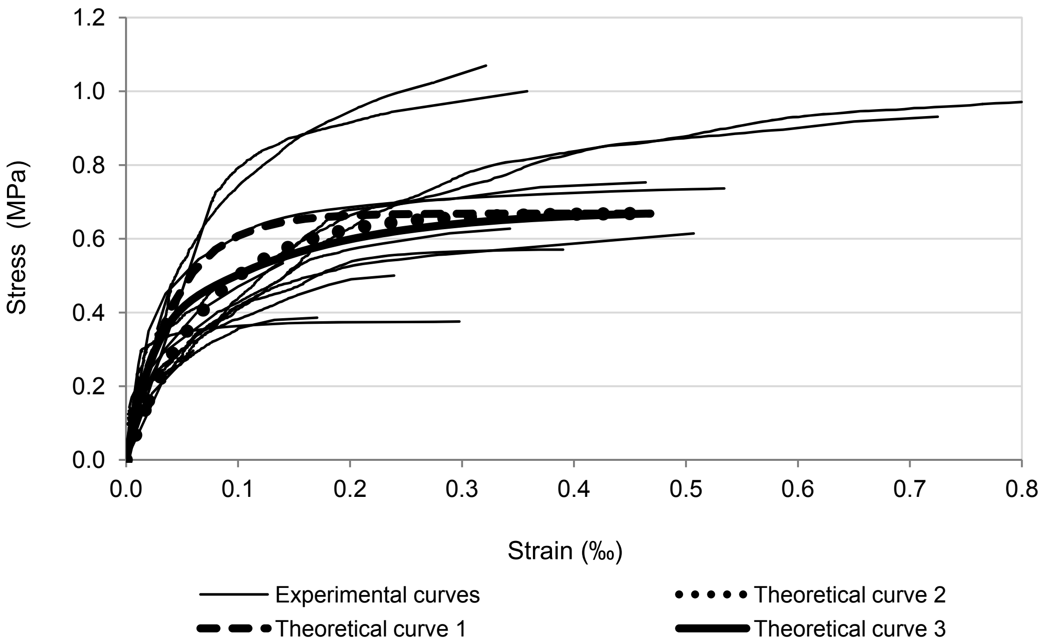

Figure 4.7: Compression stress-strain curves.

In the determination of the theoretical stress-strain curves, the following notation and units were used: 'σ_c' - compressive stress (MPa); 'ε_c' - compressive strain (‰); 'f_c' - compressive strength (or peak stress) (MPa); '$\varepsilon_{2/3f_c}$' - strain at two-thirds of peak stress (‰).

The first proposed curve presents peak stress, strain at peak stress, and secant modulus of elasticity at one-third of peak stress, equal to the respective mean values obtained in the testing of the cylindrical adobe specimens (Figure 4.7). The equation of this curve is given by:

$$\sigma_c(\varepsilon_c) = -0.6684\left(3.606\times10^{-11}\right)^{\varepsilon_c} + 0.6684 \quad (4.1)$$

Considering compressive strength as a variable, it is given by:

$$\sigma_c(\varepsilon_c) = -f_c\left(1.046\times10^{-7}\right)^{\frac{\varepsilon_c}{f_c}} + f_c \quad (4.2)$$

The initial section of this curve is a good representation of the experimental results; however, the final section shows a very sudden decrease in stiffness, with a slope of approximately zero, which is not representative of the final branch of the results obtained.

The second proposed curve presents peak stress, strain at peak stress, and secant modulus of elasticity at 80% of peak stress, equal to the respective mean values obtained in the tests conducted (Figure 4.7). The equation of this curve is given by:

$$\sigma_c(\varepsilon_c) = -0.6695\left(1.115\times10^{-6}\right)^{\varepsilon_c} + 0.6695 \quad (4.3)$$

Considering compressive strength as a variable, it is given by:

$$\sigma_c(\varepsilon_c) = -1.002 f_c\left(1.049\times10^{-4}\right)^{\frac{\varepsilon_c}{f_c}} + 1.002 f_c \quad (4.4)$$

The final section of this curve is a good representation of the experimental results; however, this curve has an initial stiffness significantly lower than the mean initial stiffness obtained in the experimental tests.

The third curve (Figure 4.7) is composed of two branches described by two exponential functions. The first function, defined for stresses in the range $\left[0,\ 2/3\,f_c\right]$, presents secant modulus of elasticity at one-third of peak stress and at two-thirds of peak stress, equal to the respective mean values obtained in the testing of the cylindrical adobe specimens. The second function, defined for stresses in the range $\left]2/3\,f_c,\ f_c\right]$, presents peak stress and strain at peak stress, equal to the respective mean values obtained in the tests conducted. At the transition point, corresponding to two-thirds of peak stress, the two functions present the same slope. The equation of this curve is given by (with $\varepsilon_{2/3f_c} = 6.41\times10^{-2}$ ‰):

$$\sigma_c\left(\varepsilon_c\right)=\begin{cases}-0.4976\left(5.096\times10^{-16}\right)^{\varepsilon_c}+0.4976, & \varepsilon_c\le\varepsilon_{2/3f_c}\\ -0.3853\left(3.794\times10^{-4}\right)^{\varepsilon_c}+0.6781, & \varepsilon_c>\varepsilon_{2/3f_c}\end{cases}\tag{4.5}$$

Considering compressive strength as a variable, it is given by (with $\varepsilon_{2/3f_c} = 9.596\times10^{-2} f_c$ (‰)):

$$\sigma_c\left(\varepsilon_c\right)=\begin{cases}-0.7445f_c\left(5.997\times10^{-11}\right)^{\frac{\varepsilon_c}{f_c}}+0.7445f_c, & \varepsilon_c\le\varepsilon_{2/3f_c}\\ -0.5764f_c\left(5.168\times10^{-3}\right)^{\frac{\varepsilon_c}{f_c}}+1.014f_c, & \varepsilon_c>\varepsilon_{2/3f_c}\end{cases}\tag{4.6}$$

This curve is a good representation of the experimental results, throughout all the range of values measured in the tests.

It is important to note that, to determine the theoretical stress-strain curves considering compressive strength as a variable, the secant stiffness values obtained for reference points (one-third, two-thirds and 80% of peak stress, and peak stress) were considered constant for different levels of compressive strength.

4.3.4. Strain at peak stress

The mean strain at peak stress, per adobe under analysis, derived from the stress-strain curves obtained in the simple compression tests performed on cylindrical specimens, is presented in Figure 4.8, with indication of mean values and coefficients of variation per house under study (calculated considering the results obtained for all the individual test specimens). Strain at peak stress ranges between 0.24‰ (*'H*20_*a*10*'*) and 1.27‰ (*'H*12_*a*03*'*), with a global mean value of 0.47‰. Results present significant variability, especially the results obtained for house *'H*12*'*.

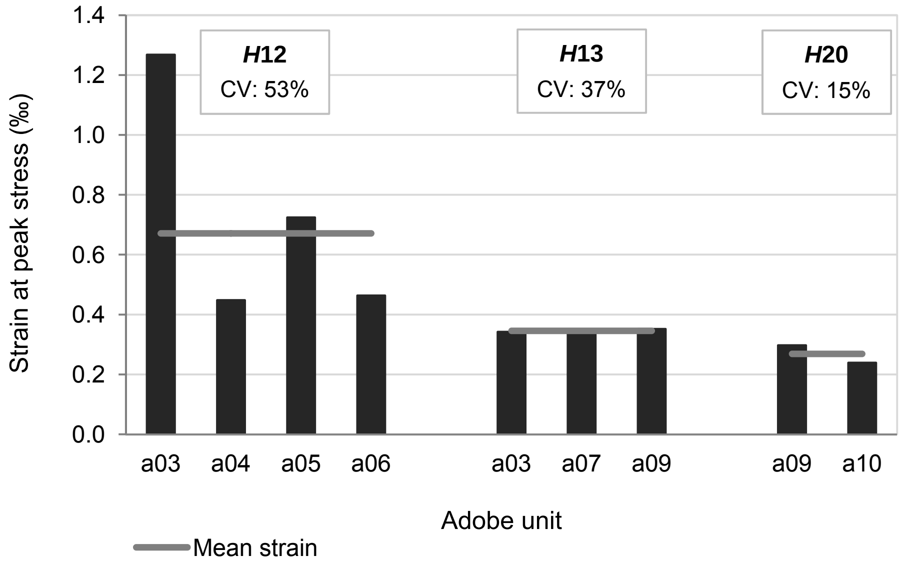

Figure 4.8: Mean strain at peak stress per adobe brick, obtained by testing cylindrical specimens in simple compression.

4.3.5. Modulus of elasticity

For cylindrical specimens, modulus of elasticity (*'E'*) was calculated as a secant modulus at one-third of peak stress (as defined in 'EN 1052-1' (CEN 1998)). Secant modulus of elasticity at peak stress ($'E_{peak}'$) was also calculated. The mean values of modulus of elasticity, calculated per adobe brick, and the global mean values and coefficients of variation corresponding to each house under study (calculated considering the results obtained for all the individual test specimens), are presented in Figure 4.9. In this figure, the secant modulus of elasticity at peak stress for adobe *'H*20_*a*08*'* is not presented, given that in the testing of the cylindrical specimen extracted from this adobe it was not possible to perform a correct measurement of the deformation at peak stress.

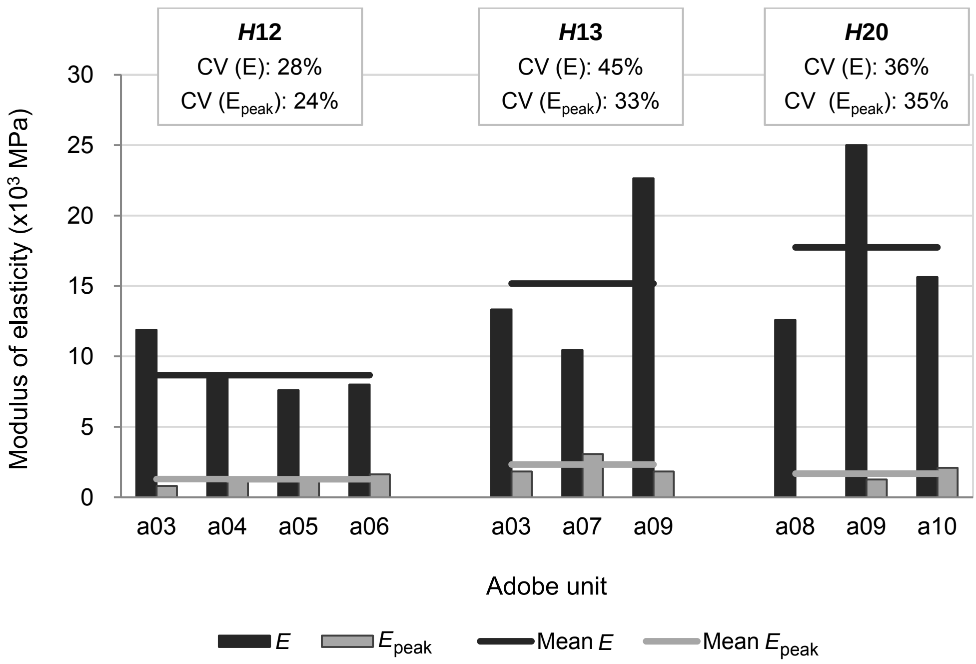

Figure 4.9: Mean modulus of elasticity per adobe brick, obtained by testing cylindrical specimens in simple compression.

The mean modulus of elasticity, calculated per adobe, ranges from 7609 MPa (*'H*12_*a*05*'*) to 25000 MPa (*'H*20_*a*09*'*), with a global mean value of 13214 MPa. The mean secant modulus of elasticity at peak stress, calculated per adobe, varies from 803 MPa (*'H*12_*a*03*'*) to 3061 MPa (*'H*13_*a*07*'*), with a global mean value of 1777 MPa. Results show significant variability, particularly the results obtained for house *'H*13*'*.

Stiffness degradation was calculated between one-third of peak stress and peak stress. Secant stiffness presents a global mean degradation of 85%. For the lowest strength class of concrete (C8/10), secant stiffness degradation calculated between one-third of peak stress and peak stress is 58% (CEN 2004), much lower than the obtained for adobe. It is interesting to note that the higher value of stiffness degradation for adobe is coherent with the tendency verified for concrete (CEN 2004) – the lower the concrete strength class, the higher the stiffness degradation.

The mean values of modulus of elasticity obtained by other authors (Gavrilovic et al. 1998; Quagliarini and Lenci 2010; Fratini et al. 2011; Eslami et al. 2012) for adobes representative of adobe construction in other regions of the world and the mean value obtained in this study are presented in Table 4.3.

Table 4.3: Modulus of elasticity of adobe specimens obtained by different authors.

Reference	Location	Adobe bricks		Test specimens	Measurement of deformations	Modulus of elasticity (MPa)
		Composition	Condition			
Present study	Portugal	Arenaceous soil and lime binder	Collected from existing constructions	Cylinders: $H \approx 0.15$ - 0.18 m $D \approx 0.08$ - 0.09 m	Performed directly on test specimens	13214
Gavrilovic et al. (1998)	Mexico	Clayey soil	Not indicated	Not indicated	Not indicated	1471
Quagliarini & Lenci (2010)	Italy	Clayey soil, straw and coarse sand, in variable proportions	New (produced for the study)	Blocks (bricks cut into 4 parts): 0.15×0.23×0.13 m^3	Measurement of the relative displacement of testing platens	98 - 211
Fratini et al. (2011)	Italy	Gravel clay, with a proportion of clay ranging from 16% up to 40%	Collected from existing constructions	Cubes: 0.05×0.05×0.05 m^3	Measurement of the relative displacement of testing platens	15 - 87
Eslami et al. (2012)	Iran	Clayey soil	New (produced for the study)	Bricks: 0.19×0.19×0.05 m^3	Not indicated	≈ 85 [a]

Notation: H - Height; D - Diameter. [a] Estimated from the stress-strain curve presented by Eslami et al. (2012).

The values of modulus of elasticity obtained for the adobe specimens tested in this study are much higher (on average, 127 times higher) than the values obtained by other authors and are closer to the lower typical values of modulus of elasticity of concrete – which normally vary between 17 GPa and 31 GPa (Cobb 2004) – and the typical values of modulus of elasticity of ceramic clay bricks – which usually range from 5 GPa to 30 GPa (Cobb 2004). This may be in part due to the differences in composition between the adobes used in Aveiro district and those tested in other regions of the world. The adobes traditionally used in Aveiro district included the addition of a significant fraction of lime binder, while the adobes tested by the other authors were made without the use of a binder. In addition, the soils typically used in the adobes of Aveiro consisted mainly of sand, sometimes including some gravel in their composition, while the adobes tested in other regions were made with clayey soils. A significant factor that must also contribute to justify this difference, however, is related to the method adopted in the measurement of deformation in compression tests. Due to the technical difficulties associated with performing the measurement of deformation directly on test specimens, this is often conducted on the load application system, and thus the additional deformation suffered by the testing device – especially in the interface with the specimen – is included in the deformation recorded. This was the method adopted in two of the studies carried out by other authors (it was not possible to verify the methods used in the other two studies) (Table 4.3). In the present study, however, deformations were measured directly on the test specimens – as explicitly recommended in 'NZS 4297:1998 Engineering design of earth buildings' (SNZ 1998a) for the measurement of deformations on earthen walls. As a result, the additional deformation suffered by the system was not measured and thus it was possible to obtain a more accurate value for the modulus of elasticity.

4.3.6. Poisson's ratio

For the determination of the Poisson's ratio, the longitudinal and transverse deformations of several of the cubes tested in simple compression were measured. However, as indicated in subsection 4.2.5, reliable measurements of transverse deformation were obtained only for one of the cubes tested (*'H*12_*a*04_*cb*05*'*), and thus Poisson's ratio was calculated only for that cube.

Poisson's ratio was calculated at one-third of peak stress, i.e. at the same level of compressive stress considered in the calculation of the modulus of elasticity. The following expression was used: $\nu = -\varepsilon_{t,1/3\,f_c} / \varepsilon_{1/3\,f_c}$, where *'ν'* is the Poisson's ratio, '$\varepsilon_{t,1/3\,f_c}$' is the transverse strain at mid-height of the specimen at one-third of peak stress, and '$\varepsilon_{1/3\,f_c}$' is the longitudinal strain at one-third of peak stress.

For the cube under analysis (*'H*12_*a*04_*cb*05*'*), a Poisson's ratio of approximately 0.10 was obtained. This value is low when compared with Poisson's ratio values of other materials. For a large part of materials, this ratio varies between 0.25 and 0.35 (Gere and Goodno 2011). For concrete, it is lower and similar to several ceramic materials (Weiss 2006), ranging approximately between 0.1 and 0.2 (Gere and Goodno 2011), with a value of 0.18 being typically considered (Weiss 2006). The Poisson's ratio obtained for the adobe cube tested coincides with the lower limit of the range of values typically considered for concrete. This value, however, is the result of a single test and is only a first indication of the Poisson's ratio of adobe. Further studies on this subject are therefore necessary.

4.3.7. Correlations

4.3.7.1. Compressive strength of cylinders and cubes

The correlation between the compressive strengths obtained by testing cylinders (‘$f_{c,cyl}$’) and cubes (‘$f_{c,cub}$’) was studied. For each adobe brick, the mean compressive strength of the cylinders extracted from that adobe was plotted against the mean compressive strength of the cubes extracted from the same adobe, and the following best-fit linear relationship was determined: $f_{c,cyl} = 0.94\, f_{c,cub}$ (Figure 4.10).

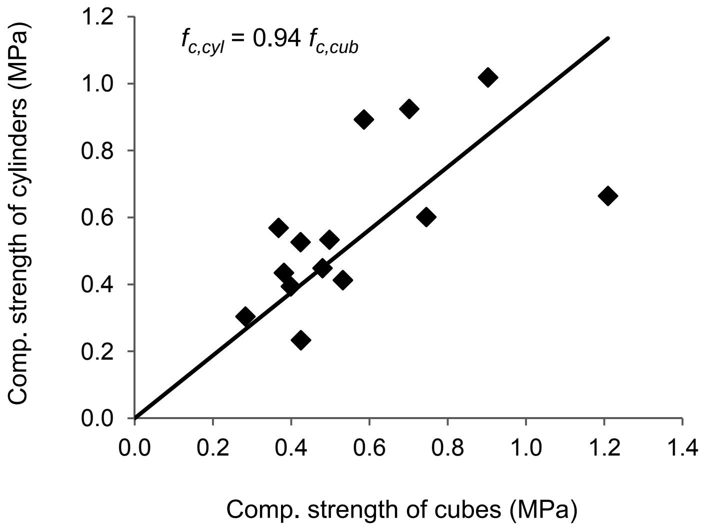

Figure 4.10: Correlation between the compressive strength of cylinders and the compressive strength of cubes.

In a simple compression test, the specimen expands laterally as the applied stress increases. Friction between the specimen and the testing platens, however, hinders lateral expansion, increasing the apparent strength of the material. This confinement effect decreases as the aspect ratio (‘k_a’ - ratio between the height and thickness of the specimen) increases. The Australian handbook (Walker 2002) and ‘NZS 4298:1998 Materials and workmanship for earth buildings’ (SNZ 1998b) present an aspect correction factor to calculate the unconfined compressive strength of adobe bricks. The values of this correction factor are similar to those derived for small walls of fired clay brick masonry (Krefeld 1938; Morel et al. 2007). According to these documents, the expected ratio between the strength of a rectangular parallelepipedic specimen with k_a = 2 and a cubic specimen (k_a = 1) is approximately equal to 0.88. For concrete, a ratio of 0.80 between the strength of a cylindrical specimen with k_a = 2 and a cubic specimen (k_a = 1) is generally considered (Domone 2001), although research indicates that this ratio depends on various factors, especially on the strength level of the material (L'Hermite 1995). It was, therefore, expected to obtain a correlation between the strength of cylindrical specimens and the strength of cubic specimens close to these ratios. The obtained ratio, however, is considerably higher ($f_{c,cyl} = 0.94\, f_{c,cub}$). Even if the confinement effect produced by the testing platens may tend to be lower for raw earth materials, when compared to concrete or ceramic materials, the small difference between the results obtained for cubes and cylinders suggests that the use of regularisation sand at the bases of specimens may contribute to minimise the confinement effect.

4.3.7.2. Splitting tensile strength and flexural tensile strength

The correlation between the splitting tensile strength of the cylinders and the flexural tensile strength of the adobe bricks tested was studied. For each adobe brick, the mean splitting strength of the cylinders extracted from the adobe halves that resulted from flexural testing was plotted against the flexural strength of the original adobe brick, and the following best-fit linear relationship was determined: $f_{t,split} = 0.30\, f_{t,flex}$ (Figure 4.11).

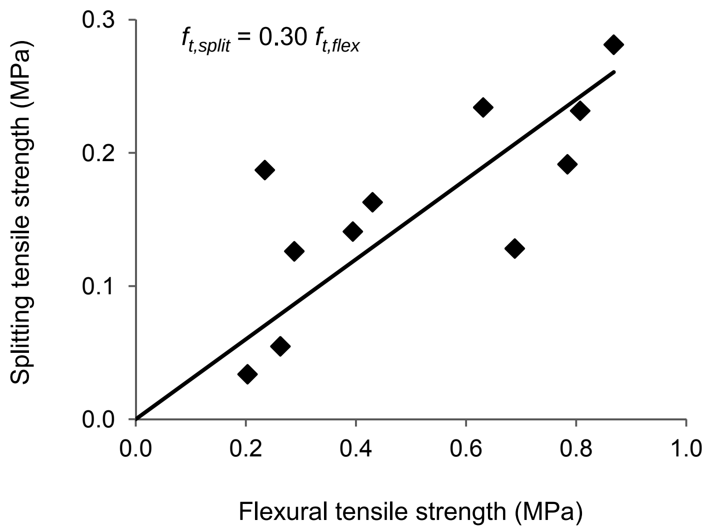

Figure 4.11: Correlation between splitting tensile strength and flexural tensile strength.

Flexural tensile strength is typically greater than splitting tensile strength. In flexural tests, failure is controlled by the strength of the material at the tension surface of the specimen, while in splitting tests failure may start at any point in the tension diametrical plane (Ozyildirim and Carino 2006). Taking into account the size effect principle, splitting tensile strength is thus expected to be lower than flexural tensile strength (Ozyildirim and Carino 2006). Another possible cause for greater flexural strength results can be related to the use of Hooke's Law in the calculation of the strength values, when the material does not actually behave according to this law (Melis et al. 1985). Finally, the ratio between the span and height of a brick tested in flexion is generally not large enough for it to behave as a perfect beam, which also contributes to greater strength values.

In several studies on the relation between splitting and flexural tensile strength carried out for concrete, the ratio between the two strengths ranges between 0.39 and 0.91, with a mean value varying between 0.60 and 0.70 (Popovics 1967; Melis et al. 1985). This relation depends, among other factors, on the dimensions of the test specimens and strength level of the material (Melis et al. 1985). The greater the strength, the greater the expected value of this ratio (Malhotra and Zoldners 1967; Melis et al. 1985). In the present study, a ratio of 0.30 was obtained. This ratio is lower than those determined for concrete, which is consistent with the demonstrated tendency, as the strength of adobe is much lower than the strength of concrete.

4.3.7.3. Modulus of elasticity and compressive strength

The correlation between the modulus of elasticity and the compressive strength obtained by testing cylinders was studied. For each cylinder, the modulus of elasticity was plotted against the compressive strength, and the following best-fit linear relationship was determined: $E = 13927\, f_{c,cyl}$ (Figure 4.12). The data points presented are considerably scattered and thus more tests are necessary to strengthen the validity of this correlation.

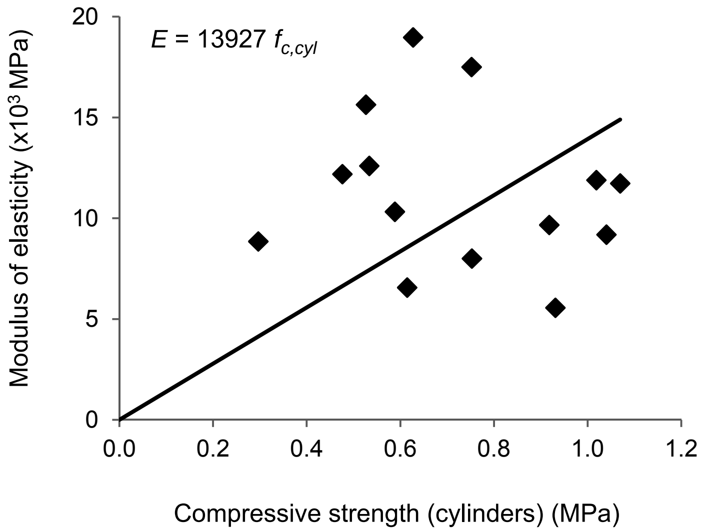

Figure 4.12: Correlation between the modulus of elasticity and compressive strength of cylinders.

4.3.7.4. Splitting tensile strength and compressive strength of cylinders

A study of the correlation between the splitting tensile strength and the compressive strength of cylindrical adobe specimens was conducted in the first experimental campaign, presented in Chapter 3. The following best-fit linear relationship was obtained: $f_{t,split} = 0.18\, f_{c,cyl}$. To strengthen this correlation, the results obtained in the present campaign were plotted in conjunction with the results of the previous campaign, and the relationship between the tensile and compressive strength was reassessed (Figure 4.13).

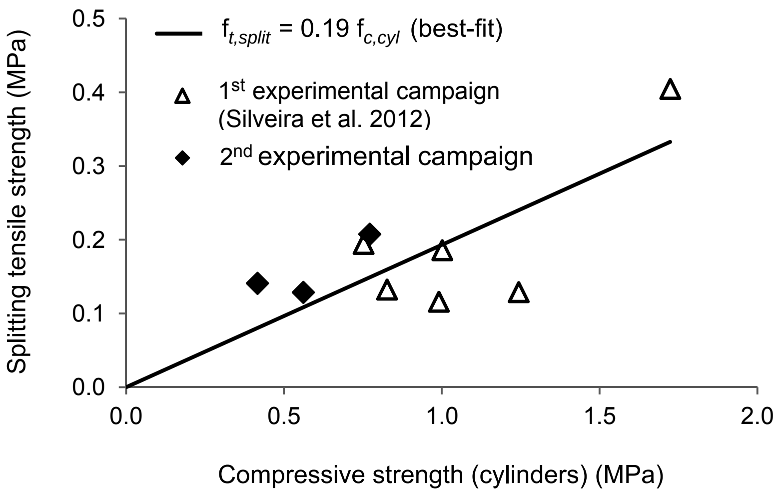

Figure 4.13: Correlation between the splitting tensile strength and compressive strength of cylinders.

In this analysis, a best-fit linear relationship of $f_{t,split} = 0.19\ f_{c,cyl}$ was obtained. The correlation determined considering all the results in conjunction is thus consistent with that previously determined.

4.4. Conclusions and final remarks

A series of mechanical tests were performed on adobe specimens extracted from adobes of existing constructions in Aveiro district, and the results obtained were presented and analysed in this chapter.

A summary of the results obtained is displayed in Table 4.4. Compressive strength obtained by testing cylindrical specimens and tensile strength obtained in splitting tests are close to the lower strength values obtained in the experimental campaign previously conducted, presented in Chapter 3. The values of modulus of elasticity obtained are much higher than those determined by other authors in the testing of adobe specimens from other regions of the world (Gavrilovic et al. 1998; Quagliarini and Lenci 2010; Fratini et al. 2011; Eslami et al. 2012). This may be due, in part, to the differences in composition between the adobes used in Aveiro district and those tested by other authors; however, it is also likely explained by the fact that, in this study, the deformation suffered by specimens during testing was measured directly on the specimens, and thus the additional deformation undergone by the testing system was not considered.

Table 4.4: Summary of the results obtained.

	Compressive strength (MPa)		Tensile strength (MPa)		Strain at peak stress (‰)	Modulus of elasticity (MPa)	Poisson's ratio
	Cubes	Cylinders	Flexural	Splitting			
Min.	0.28	0.23	0.20	0.03	0.24	7609	…
Max.	1.21	1.02	1.03	0.28	1.27	25000	…
Mean	0.54	0.58	0.56	0.16	0.47	13214	0.10 [a]

[a] Calculated using the deformations measured in one cubic specimen (*'H12_a04_cb05'*).

The results obtained vary greatly, particularly for house *'H13'*. This is consistent with the high variability of results observed in the first experimental campaign. As explained in

Chapter 3, high variability of results was expected because, in the past, the composition of adobes and production procedures could vary significantly, even for the same construction. Other authors testing adobe specimens report similar variability of results, particularly when adobes are taken from existing constructions (e.g. Meli (2005), Liberatore et al. (2006), Fratini et al. (2011)).

The correlations determined are summarised in Table 4.5. The compressive strength obtained by testing cylinders is very close to the strength obtained by testing cubes. One of the possible explanations for this is that the use of regularisation sand between the specimens and the testing platens may have contributed to lessen the confinement effect. The flexural strength values obtained, on the other hand, are significantly higher than the splitting strength values and are close to the compressive strength of cubic and cylindrical specimens (Table 4.4). This stresses the fact that flexural testing of adobe bricks can overestimate tensile strength.

Table 4.5: Summary of the correlations obtained.

Compressive strength	Tensile strength	Modulus of elasticity vs. Compressive strength
Cylinders vs. Cubes	Flexural vs. Splitting	
$f_{c,cyl} = 0.94\ f_{c,cub}$	$f_{t,flex} = 0.30\ f_{t,split}$	$E = 13927\ f_c$

As previously noted in Chapter 3, a lack of comprehensive European and international standards addressing earthen construction was noted in the planning and execution of the mechanical tests. There is a need for standardised procedures adapted to earthen construction, addressing not only new constructions but also existing ones.

Overall, the work presented contributes to the enrichment of the knowledge regarding the mechanical properties of the adobes used in traditional masonries. This work also contributes with an initial proposal for the correlation between mechanical properties evaluated with different testing procedures, allowing the possibility of selecting between procedures, according to the characteristics of adobes and existing conditions for the extraction and testing of specimens. The correlations determined, however, are not definitive and further work to validate and expand the results obtained is needed. Finally, it is important to note that the strength of a specimen depends not only on the type of test conducted and shape of the specimen but also on its dimensions – as has been observed, for example, in the testing of concrete (Ozyildirim and Carino 2006; Yazıcı and Sezer 2007) –, and thus the conclusions drawn in this study are directly applicable only to the specific dimensions of the specimens tested.

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Chapter 5

Mechanical properties and behaviour of adobe wall panels

The work reported in this chapter is presented in: Silveira, D., Varum, H., Costa, A., and Carvalho, J. (2015). “Mechanical properties and behavior of traditional adobe wall panels of the Aveiro district.” *J. Mater. Civ. Eng.,* 27(9), 04014253.

5.1. Introduction

In the study, rehabilitation, and strengthening of existing adobe constructions, knowledge regarding the mechanical properties and behaviour of adobe masonry is fundamental. The existing literature focused on adobe construction is mainly devoted to the study of the seismic behaviour of adobe structures and development of seismic retrofitting solutions (e.g. Meli (2005), Yamín et al. (2007), Dowling and Samali (2009), Tolles (2009), Blondet et al. (2011), Figueiredo et al. (2013)). The Pontifical Catholic University of Peru (PUCP), in particular, has conducted extensive research focused on adobe construction (Vargas et al. 2005). At the onset of the research conducted in PUCP, in the seventies and early eighties, one of the main research purposes was the study of the mechanical properties and behaviour of the adobe masonry traditionally used in Peru (Vargas et al. 2005). With this aim, adobe wall panels were tested in simple compression, diagonal compression, direct shear, and flexure (e.g. Blondet and Vargas (1978), Vargas and Ottazzi (1981)). Studies of the mechanical properties and behaviour of adobe masonry elements have also been carried out more recently in PUCP and by other authors (e.g. Torrealva and Acero (2005), Meli (2005), Liberatore et al. (2006), Yamín et al. (2007), Wu et al. (2013)). The majority of these studies, however, were conducted with the objective of assisting the main research program focused on the study of the seismic behaviour of adobe constructions and the development of seismic retrofitting solutions. Thus, most of these studies do not include an in-depth analysis of the results obtained (e.g. Torrealva and Acero (2005), Meli (2005), Yamín et al. (2007)). Wu et al. (2013), however, developed a more in-depth study of the mechanical behaviour of adobe masonry panels subjected to uniaxial compression, using traditional unstabilised adobe bricks and different mortar compositions.

Important research work has been conducted, but the need for more thorough studies of the mechanical properties and behaviour of adobe masonry is recognised. This need of knowledge is particularly true for lime stabilised adobe masonry. In fact, literature focused specifically on the study of the mechanical behaviour of traditional lime adobe masonry elements was not found. With the objective of contributing to this knowledge, research focused on the traditional adobe masonry of Aveiro district, in Portugal, was conducted and is presented in this chapter. Ten full-scale adobe wall panels were built in the laboratory with adobes taken from a representative house and with mortar formulated with composition similar to that traditionally used. Five wall panels were tested in simple compression and the other five were tested in diagonal compression. From these tests it was possible to determine and evaluate the stress-strain relationships, strength, stiffness, Poisson’s ratio, and damage pattern of the adobe walls. Two theoretical stress-strain curves were proposed; a comparison of the strength values obtained with the strength limits indicated in the Peruvian technical standard for adobe construction (‘Norma técnica de edificación NTE E.080 Adobe’) (ICG 2006) was performed; and comparisons of the results obtained with the results of different authors for wall panels representative of adobe construction in other countries were also carried out. The knowledge gained can be used to assist further studies focused on the behaviour of adobe structures (as in the calibration of numerical models) and also to support the rehabilitation and strengthening of existing adobe buildings.

5.2. Construction and testing of wall specimens

5.2.1. Technical standards

The existing technical standards and recommendations focused on earthen construction (e.g. SNZ (1998b), ICG (2006), RLD (2009), Walker (2002)) do not include, or only briefly include, guidelines for testing adobe masonry. Thus, the following standards, which contain detailed indications for testing masonry in general, were taken into consideration in the preparation and conduction of the tests: ‘EN 1052-1: Methods of test for masonry – Part 1: Determination of compressive strength’ (CEN 1998), in simple compression tests; and ‘E 519: Standard test method for diagonal tension (shear) in masonry assemblage’ (ASTM 2002), in diagonal compression tests.

5.2.2. Wall panels

5.2.2.1. Geometry

Ten full-scale adobe wall panels were constructed in the laboratory by experienced masons, following the procedures traditionally used in Aveiro district (Figure 5.1). The determination of the dimensions of the walls was based on the indications of ‘ASTM E 519’ (ASTM 2002), and it was ensured that these dimensions also complied with the requirements of ‘EN 1052-1’ (CEN 1998). ‘ASTM E 519’ (ASTM 2002) indicates dimensions of 1.20 x 1.20 m^2 (height x width), and the following dimensions were adopted: 1.26 x 1.26 x 0.36 m^3 (height x width x thickness). These correspond, horizontally, to two and a half adobes, vertically, to nine rows of adobes and, transversely, to one adobe. These dimensions include a render layer, with thickness of 0.02 m on the lateral faces and 0.01 m on the lower and upper faces of the walls. The joints of the wall panels had a mean thickness of 0.02 m.

According to ‘ASTM E 519’ (ASTM 2002), the specimen size of 1.20 x 1.20 m^2 was selected as the smallest that can reasonably represent a full-size masonry assemblage. It is important to note, however, that the mechanical behaviour of masonry wall panels cannot perfectly represent the behaviour of whole walls or structures. The test of whole walls and structures is thus fundamental and must also be developed in future work.

Figure 5.1: Construction of the adobe walls.

5.2.2.2. Materials

Adobes

The adobe bricks used in the construction of the wall panels were collected from a building that was undergoing demolition (house '*H*13', as defined in Chapter 4), located in the parish of Monte, in Murtosa municipality, and were in a good state of conservation. The adobes were made with arenaceous soil and air-lime binder, as is typical in Aveiro district. The mechanical properties of ten adobe bricks taken from this construction were studied in Chapter 4 (Silveira et al. 2013). The adobe bricks had the following characteristics (mean values): dimensions of 0.46 x 0.32 x 0.12 m^3; specific weight of 15 kN/m^3 (CV = 5%); compressive strength of 0.47 MPa (CV = 34%); modulus of elasticity of 15173 MPa (CV = 45%); and splitting tensile strength of 0.14 MPa (CV = 65%).

Mortar

For the joints and render, a mortar with composition similar to that traditionally used was formulated in the laboratory. A hydrated lime: ‘earth’ ratio of 1:3, in terms of bulk volume, was adopted. The ‘earth’ used consisted of a mixture of slightly clayey soil and sand (in a ratio of 1:2). The soil was collected at a site indicated by former mortar and adobe manufacturers. The mixture was classified as sand (ISO 2004), including 11% of gravel (particles with size between 2 mm and 9.5 mm) and 4% of clay and silt (particles with size lower than 0.075 mm). The mortar formulated had the following characteristics (mean values): specific weight of 17 kN/m^3 (CV = 7%); compressive strength of 0.47 MPa (CV = 24%); and flexural strength of 0.26 MPa (CV = 19%). The compressive and flexural strength were evaluated according to the indications of ‘EN 1015-11’ (CEN 1999). The mortar specimens were tested after approximately 60 days of curing. The modulus of elasticity of was not determined – however, this modulus must be close to the modulus of elasticity of the adobe bricks, since the composition, production, and curing processes used for the two materials are similar.

5.2.3. Testing

5.2.3.1. Simple compression

Five adobe wall panels were tested in simple compression, approximately 90 days after construction. The tests were displacement controlled and were conducted at a rate of 0.010 mm/s, in order to comply with the recommendations of ‘EN 1052-1’ (CEN 1998). The walls were tested in a steel frame, and the load was applied with a hydraulic actuator with a maximum load potency of 300 kN (Figure 5.2a). To distribute the load, an HEB 300 steel profile was used between the loading system and the top face of the walls (Figure 5.2a). The top face of the walls was regularised with thin sand. For the measurement of the deformation of the walls during testing, Gefran ‘PZ12-A’ rectilinear displacement transducers (with useful electrical stroke of 50 mm and independent linearity of 0.1%) were used. Three vertical displacement transducers with gage length of 0.520 m and three horizontal displacement transducers with gage length of 0.630 m were applied in each face of each wall, as illustrated in Figure 5.2b.

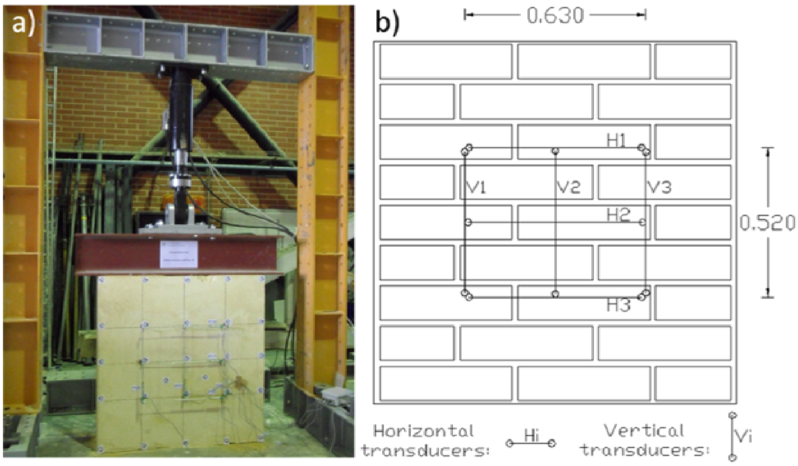


Figure 5.2: Simple compression test set-up and instrumentation layout.

5.2.3.2. Diagonal compression

Five adobe wall panels were tested in diagonal compression approximately 60 days after construction. The diagonal compression tests were conducted approximately 30 days before the simple compression tests. It was not possible to conduct the two types of test at the same curing time, due to laboratory limitations. 'EN 1015-11' (CEN 1999) indicates a curing time of 28 days for air-lime mortar prior to testing; however, the hardening process of air-lime mortars is slow and can continue during several months (Lanas and Alvarez 2003). According to Lanas and Alvarez (2003), the compressive strength of air-lime mortar (with a binder-aggregate ratio of 1:3, by volume) at 90 days is about 17% greater than the strength at 60 days (interpolating the strength values measured at 28 days and 91 days). Considering the existing practical limitations and also the great variability of strength in traditional mortars, for the purpose of the present study it is assumed that the effect of this difference in mortar strength on the results obtained is not significant.

The system composed of steel elements presented in Figure 5.3 was created for the transportation and rotation of the walls. The walls were placed in a steel frame and were wrapped in plastic film to control the projection of debris that generally accompanies sudden brittle failure (Figure 5.4a). The tests were displacement controlled and the load was applied with a hydraulic actuator with a maximum load potency of 300 kN (Figure 5.4a). For the support of the walls and for the interface between the loading system and the walls, two steel loading shoes were used, designed according to 'ASTM E 519'

(ASTM 2002) (Figure 5.5). In the second diagonal compression test, conducted on Wall 7, early cracking occurred along the lower bed joint due to the high weight of the adobe bricks. To restrain the tendency for the formation of this type of cracking, wooden planks 0.30 m wide were placed between the walls and the steel loading shoes. It is important to note that the influence of the wooden planks in the state of stress distribution in the wall panels during diagonal compression testing is not significant. The purpose of these planks is merely to avoid the early detachment of the lower row of adobe bricks.

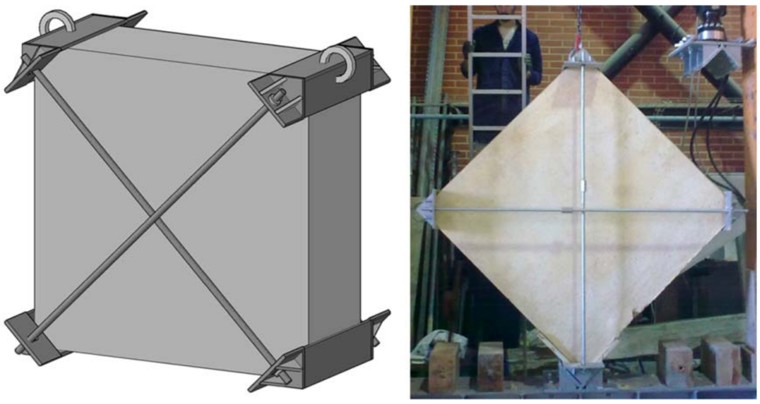


Figure 5.3: System for the transportation and rotation of the adobe walls.

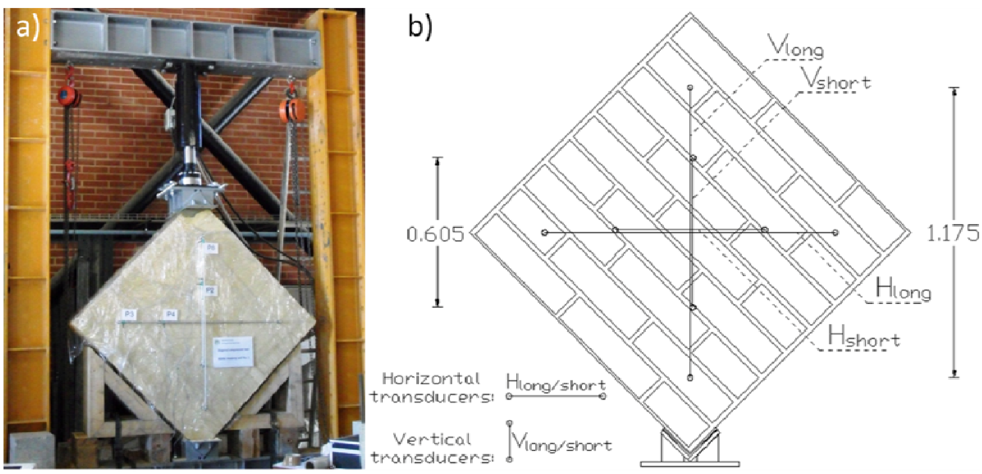


Figure 5.4: Diagonal compression test set-up and instrumentation layout.

Figure 5.5: Steel loading shoes used in the diagonal compression tests.

The first test was conducted at a rate of 0.050 mm/s, and in the following tests the rate was adjusted to 0.025 mm/s, in order to comply with the recommendations of ‘ASTM E 519’ (ASTM 2002). The deformation of the walls during testing was measured with Gefran ‘PZ12-A’ rectilinear displacement transducers (with useful electrical stroke of 50 mm and independent linearity of 0.1%). Two vertical and two horizontal displacement transducers were placed in each face of each wall, as presented in Figure 5.4b. For each direction, two different gage lengths were used: 0.605 m (‘short’ displacement transducers); and 1.175 m (‘long’ displacement transducers). The measurements of the vertical ‘long’ displacement transducers for one of the walls were lost due to intense cracking near the fixing point of the transducers, and thus the deformation values considered in the present analysis, for all the walls tested, correspond to the ‘short’ displacement transducers. A similar proportion between the gage length and the length of the diagonal of the wall (approximately 35%) has been adopted by other authors (e.g. Sousa et al. (2013)).

5.3. Results

5.3.1. Simple compression test

5.3.1.1. Stress-strain curves

The compressive stress versus horizontal and vertical strain curves obtained for the wall panels tested in simple compression are presented in Figure 5.6. The curves are represented until the point where the compressive stress decreases to approximately 80% of its maximum value. The vertical strain and the horizontal strain that correspond to this point will be referred to as ‘ultimate’.

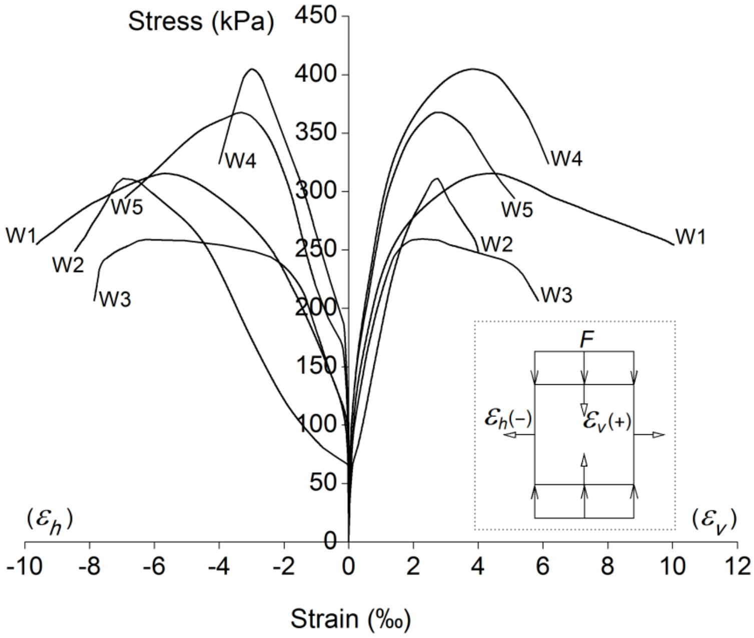


Figure 5.6: Response of the adobe walls tested in simple compression in terms of stress versus horizontal and vertical strain.

The compressive stress (‘σ_c’) was calculated by applying the following expression: $\sigma_c = \dfrac{F}{W\,t}$, where *‘F’* is the applied load, and *‘W’* and *‘t’* are the width and thickness of the wall, respectively. The weight of each wall panel was not considered in the calculation of the compressive stress. It is important to note, however, that the weight of each wall (about 8.3 kN), corresponds to approximately 6% of the mean maximum load applied (146 kN).

The horizontal strain (‘ε_h’) and the vertical strain (‘ε_v’) were calculated using the expressions $\varepsilon_h = -\Delta H / g$ and $\varepsilon_v = \Delta V / g$, respectively, where ‘$\Delta H$’ is the horizontal extension, ‘ΔV’ is the vertical shortening, and ‘g’ is the gage length. The horizontal and vertical strain correspond to the mean of the strain values calculated using the measurements of the three horizontal displacement transducers and the three vertical displacement transducers, respectively.

From the compressive stress-strain curves (Figure 5.6), it can be observed that:

- At the beginning of loading the walls display a nearly linear behaviour, up to a mean stress value of approximately 35% of maximum stress;
- In the quasi-linear phase, horizontal deformation is lower than vertical deformation but, after this phase, horizontal deformation becomes greater, reaching values close to or even higher than those in the vertical direction;
- The walls suffer brittle failure, i.e. when the maximum stress is reached, the walls rapidly develop failure, with small deformations.

5.3.1.2. Compressive strength

The compressive strength of the walls varies between 258 kPa and 405 kPa, with a mean value of 331 kPa and coefficient of variation of 17% (Table 5.1). The mean compressive strength obtained is approximately 70% of the mean compressive strength of the adobes and mortar used in the construction of the walls. This is consistent with the fact that the strength of adobe masonry, in addition to depending on the strength and behaviour of its constituent materials (adobe bricks and mortar), also depends on other factors, such as the quality of adhesion between adobes and joint mortar (Bosiljkov et al. 2005). In fact, in subsection 5.3.1.7, devoted to the analysis of the damage on the walls, it is observed that cracking is predominantly initiated in the interface between adobe bricks and head joint mortar. In this way, head joints may contribute to a lower compressive strength of the adobe walls. It is important to note that, in general, adobe wall panels tested in other studies also have compressive strength lower than that of the adobe bricks (e.g. San Bartolomé and Pehovaz (2005), Yamín et al. (2007), Wu et al. (2013)), as will be further discussed in subsection 5.3.4.

Table 5.1: Results obtained in simple compression tests.

Mechanical properties	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Mean	CV (%)
Compressive strength (kPa)	315	311	258	405	368	331	17
Vertical strain at peak stress (‰)	4.11	2.76	1.96	3.67	2.78	3.06	28
Ultimate vertical strain (‰)	… [a]	4.00	5.85	6.17	5.11	5.28	18
Modulus of elasticity (MPa)	782	… [a]	684	741	821	757	8
Secant modulus of elasticity at peak stress (MPa)	… [a]	113	132	110	132	122	10
Secant modulus of elasticity at the 'endpoint' (MPa)	25	62	35	52	58	47	34
Poisson's ratio	0.29	0.19	0.07	0.04	0.22	0.16	63

[a] The values that are not presented were identified as outliers by applying Peirce's criterion and were excluded from the analysis.

'NZS 4297' (SNZ 1998a) indicates (for 'special grade earth construction') that the compressive strength of adobe walls can be taken as the unconfined compressive strength obtained in the testing of the individual adobe bricks. The results obtained in the present study show that this indication of 'NZS 4297' (SNZ 1998a) should be considered with caution, since the strength of an adobe masonry wall can be significantly lower than the strength of the adobe bricks used in its construction.

5.3.1.3. Vertical strain at peak stress and ultimate vertical strain

The ultimate vertical strain ranges from 4.00‰ to 6.17‰, with a mean value of 5.28‰ and coefficient of variation of 18% (Table 5.1). The vertical strain at peak stress has a mean value of 3.06‰, which is 58% of the mean ultimate vertical strain, varying between 1.96‰ to 4.11‰, with coefficient of variation of 28% (Table 5.1).

5.3.1.4. Modulus of elasticity

The secant modulus of elasticity ('E_{sec}'), for a given compressive stress level, was calculated using the following expression: $E_{sec} = \sigma_c / \varepsilon_v$. The secant modulus of elasticity at one-third of peak stress varies between 684 MPa and 821 MPa, with a mean value of 757 MPa and coefficient of variation of 8% (Table 5.1). The secant modulus of elasticity at one-third of peak stress corresponds to the modulus of elasticity of masonry as defined by 'EN 1052-1' (CEN 1998), thus, hereinafter, for the sake of simplicity, it will be referred to simply as ‘modulus of elasticity’ (‘E’). The secant modulus of elasticity at peak stress and the secant modulus of elasticity at the ‘endpoint’ (i.e. at the point where the compressive stress decreases to approximately 80% of its maximum value) were also calculated, having mean values of 122 MPa and 47 MPa and coefficients of variation of 10% and 34%, respectively (Table 5.1).

The mean modulus of elasticity obtained is 5% of the mean modulus of elasticity of the adobes used in the construction of the wall panels. The mean secant modulus of elasticity at peak stress of the walls is also 5% of the mean secant modulus of elasticity at peak stress of the adobes. The stiffness of the adobe masonry walls is thus markedly lower than the stiffness of the adobes. Examples of ratios for adobe masonry obtained by other authors in tests conducted in comparable conditions were not found in the literature. For other materials, this ratio is considerably variable. For example, in a study of granitic stone masonry with large irregular blocks, as expected, the ratio obtained is very low (approximately 1%) (Almeida et al. 2012). In a study of hand moulded burnt clay brick masonry, the ratio obtained is greater and varies significantly with the quality of the mortar used (approximately from 40% to 70%) (Kaushik et al. 2007).

The correlation between the modulus of elasticity (‘E’) and the compressive strength (‘f_c’) of the walls was studied. For each wall, the mean modulus of elasticity was plotted against the respective mean compressive strength, and the best-fit linear correlation was determined (Figure 5.7). The following correlation was obtained: $E = 2203 f_c$. ‘NZS 4297’ (SNZ 1998a) indicates (for ‘special grade earth construction’) that the modulus of elasticity for earthen walls shall be taken as $300 f_c$. This standard, however, also indicates that for soil cement with silty, sandy, or gravely soils, this formula will give a low estimate. For these soils, according to this standard, the modulus of elasticity of soil cement walls can assume values as high as 20 GPa. The adobes of the present study are stabilised with lime and are made with sandy soils, sometimes with some gravel in their composition; thus, the modulus of elasticity of the masonry tested must lie somewhere between the modulus of elasticity of the walls made with unstabilised adobes and that of the walls made with soil-cement adobes, addressed by ‘NZS 4297’ (SNZ 1998a). This helps to explain why the coefficient of the relationship obtained in the present study

($E = 2203 f_c$) is significantly higher than that indicated in 'NZS 4297' for unstabilised earthen walls ($E = 300 f_c$). It is important to note, however, that given the low number of data points considered in this analysis, further experimental work is fundamental to validate and strengthen the correlation obtained.

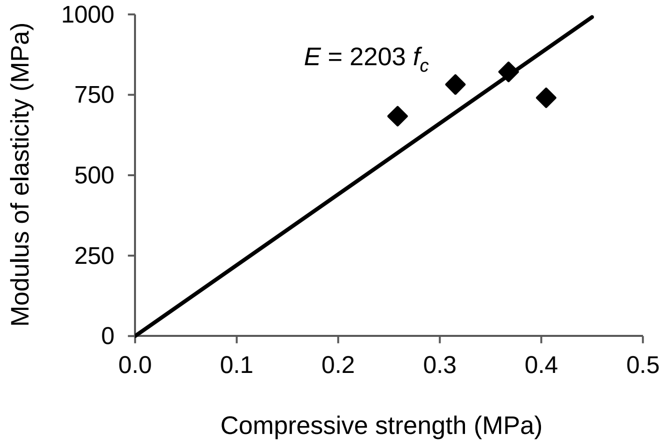


Figure 5.7: Correlation between the modulus of elasticity and compressive strength of the adobe walls.

5.3.1.5. Poisson's ratio

For some of the wall panels tested, the values of Poisson's ratio obtained at one-third and even at 25% of peak stress were unrealistic (greater than 0.5), which indicates that for these levels of stress some cracking has already occurred. Poisson's ratio ('v') was thus calculated at 20% of maximum stress, by applying the following expression: $v = -\varepsilon_{h,0.2 f_c} / \varepsilon_{v,0.2 f_c}$, where '$\varepsilon_{h,0.2 f_c}$' is the horizontal (transverse) strain at 20% of peak stress, and '$\varepsilon_{v,0.2 f_c}$' is the vertical (longitudinal) strain at 20% of peak stress. The values of Poisson's ratio obtained range from 0.04 to 0.29, with a mean value of 0.16 and coefficient of variation of 63% (Table 5.1).

The mean Poisson's ratio obtained (0.16) is greater than the Poisson's ratio determined for a single adobe specimen (0.10) in the experimental study presented in Chapter 4. The mean value obtained, however, is coherent with what is expected for this material. In a recent study of the mechanical properties in simple compression of nine adobe masonry prisms, built with adobe bricks and mortars produced with a mixture of clayey soil and sand, conducted by Wu et al. (2013), the following approximate values of Poisson's ratio, calculated for 20% of maximum stress, were obtained: 0.09, 0.18 and 0.28 (mean of 0.18). Despite the fact that the characteristics of the materials and test conditions adopted are different, it can be noted that the interval of values and the mean value obtained are close to those determined in the present study.

For many materials, Poisson's ratio ranges from 0.25 to 0.35 (Gere and Goodno 2011). For concrete, it is lower and similar to various ceramic materials (Weiss 2006), varying approximately between 0.10 and 0.20 (Gere and Goodno 2011), with a typical value of 0.18 (Weiss 2006). Thus, the mean Poisson's ratio obtained in the present study is near the centre of the interval typically considered for concrete and different ceramic materials.

5.3.1.6. Theoretical stress-strain curves

Two theoretical stress-strain curves were determined and are proposed as approximate representations of the stress-strain curves obtained for the wall panels tested in simple compression (Figure 5.8). As mentioned in Chapter 4, the determination of an adequate stress-strain constitutive law is important as it conveys essential information about the properties and mechanical behaviour of adobe masonry (Gere and Goodno 2011). A constitutive law can support the numerical modelling of the behaviour of adobe masonry and can also assist the validation of the results obtained in experimental tests. The constitutive laws determined can be applied directly in numerical models, or simpler laws, based on these more complex laws, can be used. The softening branch determined allows the representation of the behaviour of the material beyond the point of maximum strength. Studies indicate, however, that the softening behaviour under uniaxial compression of materials such as concrete, soil, or rocks, determined in conventional laboratory tests, is very dependent on factors like specimen dimensions, boundary conditions, and testing device characteristics (e.g. Read and Hegemier (1984), Van Vliet and Van Mier (1995)). Thus, it is essential to note that the softening branch determined is only valid for the specific test conditions and specimen characteristics used in the present work.

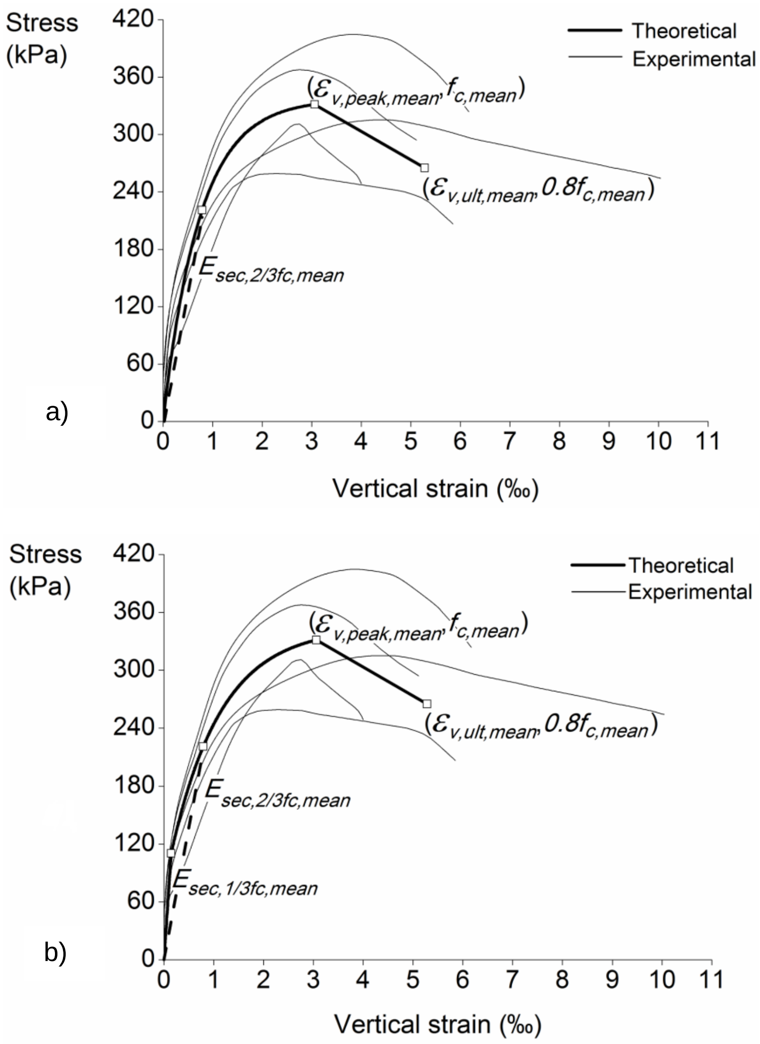


Figure 5.8: Theoretical compression stress-strain curves.

In this subsection the following units are used: permillage (‰) for vertical strain; kilopascal (kPa) for compressive stress; and megapascal (MPa) for modulus of elasticity. The first theoretical stress-strain curve proposed is composed of two parts (Figure 5.8a):

i) The first part, defined in the interval $\left[0,\ \varepsilon_{v,peak}\right]$ (where '$\varepsilon_{v,\,peak}$' is the vertical strain at peak stress), is expressed by an exponential function; this curve presents secant modulus of elasticity at two-thirds of peak stress equal to the corresponding mean secant modulus of elasticity of the walls tested, and it ends at a point with coordinates that correspond to the mean peak stress and mean vertical strain at peak stress of the walls tested;

ii) The second part, defined in the interval $\left]\varepsilon_{v,peak}, \varepsilon_{v,ult}\right]$ (where '$\varepsilon_{v,ult}$' is the ultimate vertical strain), is linear and ends at a point with coordinates that correspond to 80% of the mean peak stress and the mean ultimate vertical strain of the walls tested.

This first theoretical stress-strain curve is defined by the following equation (with $\varepsilon_{v,peak} = 3.06‰$ and $\varepsilon_{v,ult} = 5.28‰$):

$$\sigma_c(\varepsilon_v) = \begin{cases} -336.9 \times 0.2590^{\varepsilon_v} + 336.9, & 0 < \varepsilon_v \le \varepsilon_{v,peak} \\ -29.79\varepsilon_v + 422.6, & \varepsilon_{v,peak} < \varepsilon_v \le \varepsilon_{v,ult} \end{cases} \quad (5.1)$$

The initial section of the second theoretical stress-strain curve proposed provides a better representation of the experimental results. This curve is composed of three parts (Figure 5.8b):

i) The first part, defined in the interval $\left[0, \varepsilon_{v,1/3f_c}\right]$ (where '$\varepsilon_{v,1/3f_c}$' is the vertical strain at one-third of peak stress), is linear with a slope equal to the mean secant modulus of elasticity at one-third of peak stress of the walls tested;

ii) The second part, defined in the interval $\left]\varepsilon_{v,1/3f_c}, \varepsilon_{v,peak}\right]$, is expressed by an exponential function; this curve presents secant modulus of elasticity at two-thirds of peak stress equal to the corresponding mean secant modulus of elasticity of the walls tested, and it ends at a point with coordinates that correspond to the mean peak stress and mean vertical strain at peak stress of the walls tested;

iii) The third part, defined in the interval $\left]\varepsilon_{v,peak}, \varepsilon_{v,ult}\right]$, is linear and ends at a point with coordinates that correspond to 80% of the mean peak stress and the mean ultimate vertical strain of the walls tested.

This second stress-strain curve is defined by the following equation (with $\varepsilon_{v,1/3f_c} = 0.146‰$, $\varepsilon_{v,peak} = 3.06‰$, and $\varepsilon_{v,ult} = 5.28‰$):

$$\sigma_c\left(\varepsilon_v\right)=\begin{cases}757.0\,\varepsilon_v,\ 0\le\varepsilon_v\le\varepsilon_{v,1/3\,f_c}\\ -270.5\times 0.3707^{\varepsilon_v}+344.5,\ \varepsilon_{v,1/3\,f_c}<\varepsilon_v\le\varepsilon_{v,\,peak}\\ -29.79\,\varepsilon_v+422.6,\ \varepsilon_{v,\,peak}<\varepsilon_v\le\varepsilon_{v,ult}\end{cases} \tag{5.2}$$

Based on this second theoretical stress-strain relationship, two different stress-strain laws were defined considering compressive strength as a variable. Firstly, the values of secant modulus of elasticity for reference points (one-third and two-thirds of peak stress, peak stress, and 80% of peak stress) were considered constant for different levels of compressive strength. The following expression (with $\varepsilon_{v,1/3\,f_c}=f_c/2271$ (‰), $\varepsilon_{v,\,peak}=f_c/108.4$ (‰), and $\varepsilon_{v,ult}=f_c/62.75$ (‰)) was determined:

$$\sigma_c\left(\varepsilon_v\right)=\begin{cases}757.0\,\varepsilon_v,\ 0\le\varepsilon_v\le\varepsilon_{v,\,1/3\,f_c}\\ -0.8160\,f_c\left(1.336\times 10^{-143}\right)^{\frac{\varepsilon_v}{f_c}}+1.039\,f_c,\ \varepsilon_{v,1/3\,f_c}<\varepsilon_v\le\varepsilon_{v,\,peak}\\ -29.79\,\varepsilon_v+1.275\,f_c,\ \varepsilon_{v,\,peak}<\varepsilon_v\le\varepsilon_{v,ult}\end{cases} \tag{5.3}$$

Some examples of the application of this expression for different values of compressive strength are presented in Figure 5.9a.

Secondly, the values of vertical strain for reference points (one-third and two-thirds of peak stress, peak stress, and 80% of peak stress) were considered constant for different levels of compressive strength. The following expression (with $\varepsilon_{v,1/3\,f_c}=0.146$ ‰, $\varepsilon_{v,\,peak}=3.06$ ‰, and $\varepsilon_{v,ult}=5.28$ ‰) was determined:

$$\sigma_c\left(\varepsilon_v\right)=\begin{cases}2.284\,f_c\,\varepsilon_v,\ 0\le\varepsilon_v\le\varepsilon_{v,1/3\,f_c}\\ -0.8160\,f_c\times 0.3707^{\varepsilon_v}+1.039\,f_c,\ \varepsilon_{v,1/3\,f_c}<\varepsilon_v\le\varepsilon_{v,\,peak}\\ -0.08988\,f_c\varepsilon_v+1.275\,f_c,\ \varepsilon_{v,\,peak}<\varepsilon_v\le\varepsilon_{v,ult}\end{cases} \tag{5.4}$$

Some examples of the application of this expression are presented in Figure 5.9b.

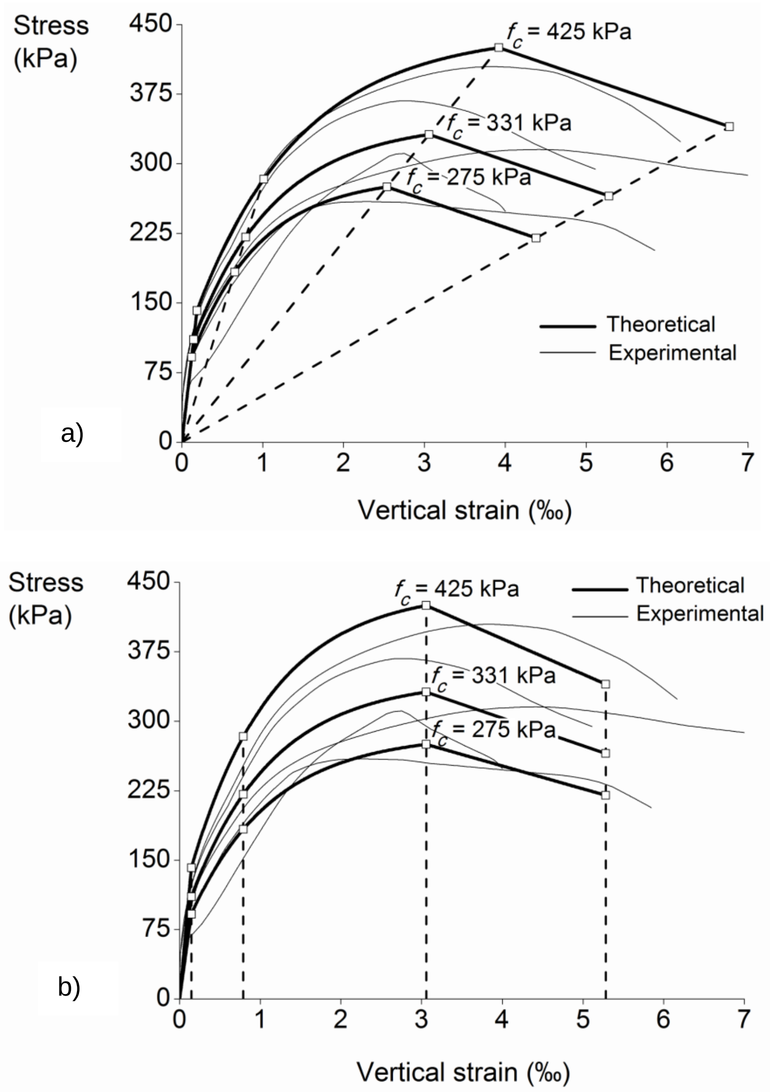
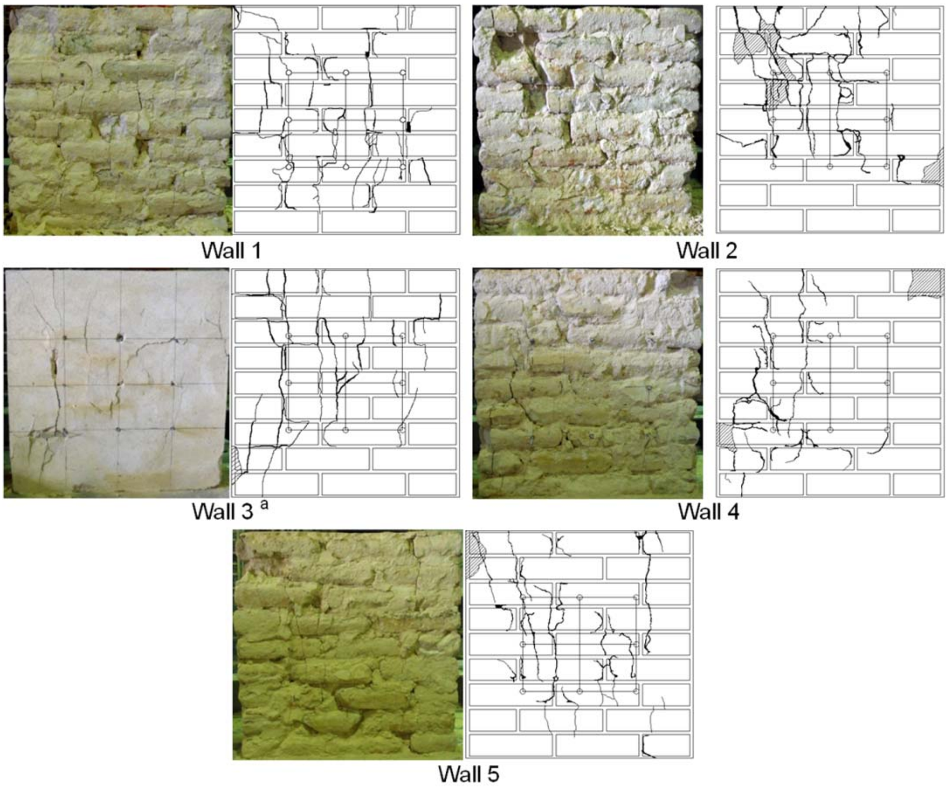


Figure 5.9: Examples of theoretical stress-strain curves with different values of compressive strength, by fixing (for reference points): a) the values of secant modulus of elasticity; b) the values of vertical strain.

5.3.1.7. Damage pattern

The damage pattern on the wall panels (without render) is presented in Figure 5.10. Failure of walls occurred mainly with the development of vertical splitting cracks. This failure pattern is typical in the uniaxial compression testing of masonry and has been observed by other authors in the testing of adobe masonry and other types of masonry (e.g. Yamín et al. (2007), Wu et al. (2013), Kaushik et al. (2007), Almeida et al. (2012)). As can be observed in Figure 5.10, the orientation of cracking is varied but predominantly vertical, and the distribution of damage is scattered along the height of the walls. Vertical cracks are predominantly aligned with the head joints of masonry, following along these joints and cutting through the intermediate adobe bricks. Damage was initiated mainly in head joints, in the interface between adobes and joint mortar. These joints, thus, are potential areas of weakness in adobe masonry subjected to simple compression. The fact that the compressive strength of the adobe masonry wall panels is lower than the compressive strength of the constituent materials (adobe bricks and mortar) is consistent with this observation.



[a] No picture of Wall 3 without plaster is available, thus the picture of the wall with plaster is presented.

Figure 5.10: Damage pattern on the adobe walls tested in simple compression.

The first visible cracking in walls occurred at a mean compressive stress corresponding to 62% of the maximum strength. This level of stress is significantly greater than the level of stress corresponding to the loss of linear behaviour (approximately 35% of maximum stress, as previously discussed), which means that minor cracking began well before it was visually observable.

5.3.2. Diagonal compression test

5.3.2.1. Stress-strain curves

The shear stress versus horizontal and vertical strain curves obtained for the wall panels tested in diagonal compression are presented in Figure 5.11. The curves are represented until the point where the shear stress decreases to about 80% of its maximum value. The shear strain that corresponds to this point will be referred to as 'ultimate'. The strain values represented correspond to the deformation measured by the 'short' displacement transducers (gage length of 0.605 m), as indicated previously.

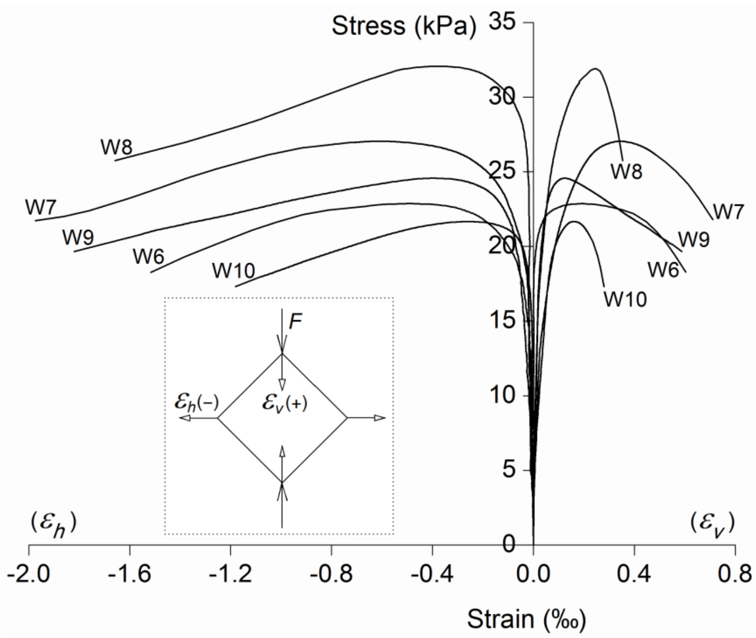


Figure 5.11: Response of the adobe walls tested in diagonal compression in terms of shear stress versus horizontal and vertical strain.

The shear stress ('τ') was calculated by applying the following expression: $\tau = \dfrac{0.707\,F}{W\,t}$. This shear stress corresponds to the principal tensile stress in the centre of the wall panel, considering that the panel is subjected to pure shear and assuming isotropic elastic material properties. The weight of each wall panel was not considered in the calculation of the shear stress. The weight of each wall (about 8.3 kN), however, corresponds to approximately 52% of the mean maximum load applied (16 kN). This is a significant percentage, but, given the complexity of the stress distribution in the wall panel caused by its self-weight, for the purposes of this analysis it was opted to calculate the shear strength considering only the applied load, as indicated in 'ASTM E 519' (ASTM 2002). Nevertheless, a simple estimate of the influence of the self-weight in the shear strength obtained was calculated. Considering failure starting at the centre of the wall, it was assumed an additional load equal to half the weight of the wall. This additional load leads to an increase in shear strength that ranges from 21% to 31%. Further studies specifically focused on the determination of the shear strength of adobe wall panels independent of the effect of self-weight, including a dedicated experimental testing campaign combined with numerical modelling, are necessary.

From the shear stress-strain curves (Figure 5.11), it can be observed that:

- In the initial phase of loading, until a mean stress value of approximately 75% of peak stress, the walls show quasi-linear behaviour;
- In the quasi-linear phase, horizontal deformation is generally lower than vertical deformation, but after peak stress, horizontal deformation becomes significantly greater;
- The walls suffer brittle failure, i.e. when the maximum capacity is reached, a sudden reduction of strength occurs, with small deformations.

5.3.2.2. Shear strength

The shear strength obtained varies between 22 kPa and 32 kPa, with a mean value of 26 kPa and coefficient of variation of 16% (Table 5.2). The mean shear strength is 8% of the mean compressive strength obtained in the simple compression tests. This percentage is very close to the relation indicated in 'NZS 4297' (SNZ 1998a) for 'special grade earth construction' (7%).

Table 5.2: Results obtained in diagonal compression tests.

Mechanical properties	Wall 6	Wall 7	Wall 8	Wall 9	Wall 10	Mean	CV (%)
Shear strength (kPa)	23	27	32	25	22	26	16
Shear strain at peak stress (‰)	0.54	0.76	0.61	0.42	0.35	0.54	30
Ultimate shear strain (‰)	2.11	2.72	2.01	2.41	1.46	2.14	22
Modulus of rigidity (MPa)	… [a]	336	497	388	432	413	16
Secant modulus of rigidity at peak stress (MPa)	42	36	52	59	61	50	22
Secant modulus of rigidity at the ‘endpoint’ (MPa)	9	8	13	8	12	10	23

[a] The value that is not presented was identified as an outlier by applying Peirce's criterion and was excluded from the analysis.

5.3.2.3. Shear strain at peak stress and ultimate shear strain

The shear strain (‘γ’) was determined by applying the following expression: $\gamma = (\Delta V + \Delta H) / g$. The vertical and horizontal strain values used correspond to the deformation measured by the ‘short’ displacement transducers (gage length of 0.605 m). The ultimate shear strain obtained varies between 1.46‰ and 2.72‰, with a mean value of 2.14‰ and coefficient of variation of 22% (Table 5.2). The shear strain at peak stress has a mean value of 0.54‰, which is 25% of the mean ultimate shear strain, and varies between 0.35‰ and 0.76‰ with coefficient of variation of 30% (Table 5.2).

5.3.2.4. Modulus of rigidity

The secant modulus of rigidity (‘G_{sec}’), for a given shear stress level, was calculated using the following expression: $G_{sec} = \tau / \gamma$. As indicated before, the shear strain was calculated with the deformation values measured by the ‘short’ displacement transducers (gage length of 0.605 m). The secant modulus of rigidity at one-third of peak stress – which hereinafter, for the sake of simplicity, will be referred to simply as ‘modulus of rigidity’ (‘G’) – varies between 336 MPa and 497 MPa, with a mean value of 413 MPa and coefficient of variation of 16% (Table 5.2). The secant modulus of rigidity at peak stress and the secant modulus of rigidity at the ‘endpoint’ (i.e. at the point where the compressive stress decreases to approximately 80% of its maximum value) have mean values of 50 MPa and 10 MPa and coefficients of variation of 22% and 23%, respectively (Table 5.2).

The mean modulus of rigidity is 55% of the mean modulus of elasticity obtained in the simple compression tests. Considering the relation between modulus of rigidity and modulus of elasticity for isotropic linear elastic materials, given by $G = \frac{E}{2(1+\nu)}$, and the experimental Poisson's ratio values determined for each wall, a relation between modulus of rigidity and modulus of elasticity varying between $G = 0.39E$ and $G = 0.48E$ was obtained. The coefficient of the experimental relation determined is thus greater than the coefficient of the upper limit of the interval obtained considering the expression for isotropic linear elastic materials. The relation proposed in 'Eurocode 6' (CEN 2005) is $G = 0.40\,E$, which corresponds to a Poisson's ratio of 0.25. It is important to note that this comparison, although interesting, has limitations. In fact, adobe masonry is not a perfect isotropic linear elastic material, and thus the experimental parameters determined are not exactly comparable to those determined by applying the expression given above.

5.3.2.5. Influence of the gage length in the results obtained

The deformations considered in the present analysis were measured by the 'short' displacement transducers, which have a gage length of 0.605 m. The influence of the gage length in the results obtained was assessed by comparing the parameters calculated with the information given by the 'long' (gage length of 1.175 m) and 'short' displacement transducers. The mean shear strain at peak stress calculated using the data provided by the 'long' displacement transducers is 50% of the mean shear strain at peak stress determined using the data given by the 'short' displacement transducers. The mean modulus of rigidity calculated with the information provided by the 'short' displacement transducers is 34% of the mean modulus of rigidity determined using the data given by the 'long' displacement transducers. It is also relevant to note that the coefficient of variation of the strain values obtained with the 'short' displacement transducers is significantly lower than that of the strain values obtained with the 'long' displacement transducers (17% lower in the case of shear strain at one-third of peak stress, and 55% lower in the case of shear strain at peak stress). The influence of the gage length in the results obtained can thus be very significant.

This is mainly justified by the fact that the deformation caused by localised cracking corresponds to higher strain values for smaller gage lengths and also by the fact that cracking tends to be more intense in the central area of the wall. Other factors may also influence the difference in results obtained, including, for example, the different number of joints that are covered by the 'long' and 'short' gage lengths.

5.3.2.6. Damage pattern

The damage pattern on the wall panels tested in diagonal compression is presented in Figure 5.12. On Walls 7, 8 and 9 there was initial cracking caused by the weight of the lower adobe rows. Given that this damage is not a result of the diagonal compression test, it was not represented in the damage drawings.

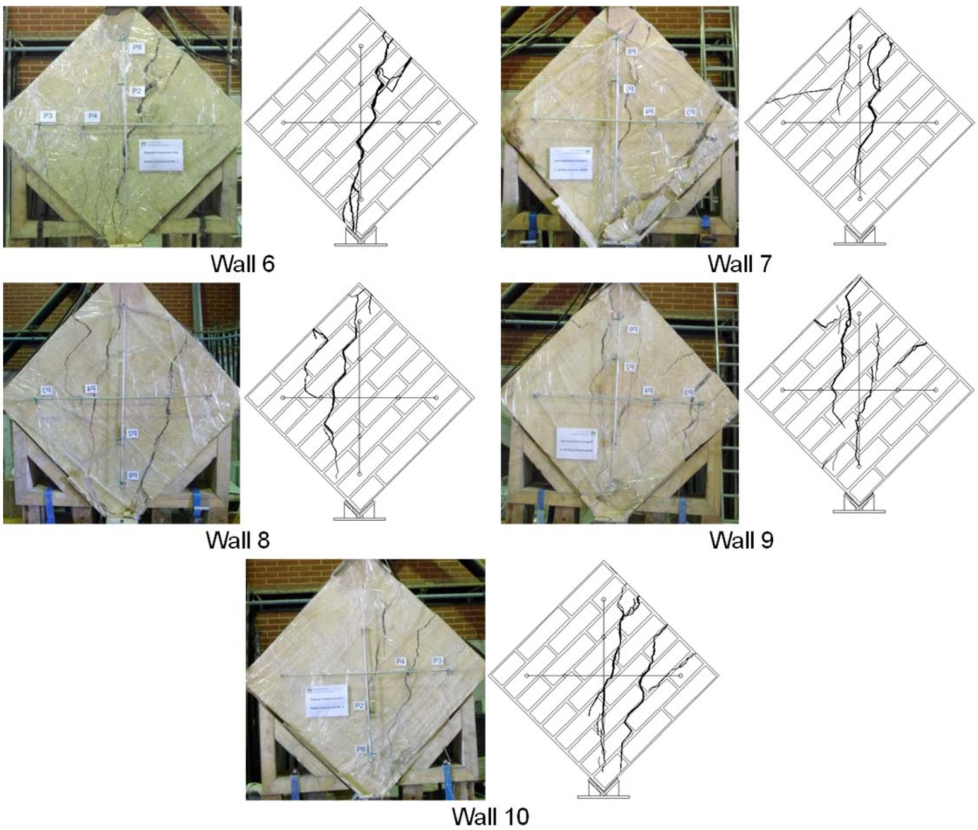


Figure 5.12: Damage pattern on the adobe walls tested in diagonal compression.

Failure of walls occurred with the development of a large central vertical crack or, in other cases, two dominant central vertical cracks. This failure pattern is typical in the diagonal compression testing of masonry and has been recorded by other researchers in the testing of adobe masonry and other types of masonry (e.g. San Bartolomé and Pehovaz (2005), Yamín et al. (2007), Brignola et al. (2009)). As can be observed in Figure 5.12, cracks are not perfectly vertical but are slightly inclined. Cracks follow along mortar joints in a significant part of their path, forming a stepped pattern, and in the remaining parts cut directly through adobe bricks – mortar joints may thus be areas of weakness in adobe masonry subjected to diagonal compression.

5.3.3. Comparison with normative limits

The maximum values of compressive strength (405 kPa) and shear strength (32 kPa) obtained in the present study are considerably lower than the strength limits indicated in 'NTE E.080' (ICG 2006) for adobe masonry (compressive strength limit: 785 kPa, considering a maximum allowable stress of 196 kPa; shear strength limit: 61 kPa, considering a maximum allowable stress of 25 kPa). This is coherent with the results of the previous experimental study, presented in Chapter 3, focused on the mechanical characterisation of adobe bricks collected from existing constructions in Aveiro district. In this previous study, it was observed that the compressive and tensile strength values of the adobe bricks were, in general, lower than the minimum limits indicated in different earthen construction technical standards (Silveira et al. 2012). Earthen construction standards (e.g. SNZ (1998b), ICG (2006)) focus on new earthen buildings, while the adobes tested previously, as well as those used in the construction of the wall panels, were taken from existing old constructions. Current requirements imposed by these standards aim at the production of adobe masonry with improved characteristics, and generally existing adobe masonry cannot meet these requirements.

5.3.4. Comparison with the results obtained by other authors

Table 5.3 presents a summary of the mean results obtained by other authors (Meli 2005; San Bartolomé and Pehovaz 2005; Torrealva and Acero 2005; Liberatore et al. 2006; Yamín et al. 2007; Wu et al. 2013) in simple compression and diagonal compression tests conducted on adobe wall panels representative of adobe construction in other countries, together with a summary of the mean results obtained in the present study. For a more rigorous comparison of the compressive strength obtained by the different authors, the unconfined compressive strength was calculated by applying the aspect ratio factor as defined by ‘NZS 4298’ (SNZ 1998b). Because the test procedures and characteristics of test specimens used by the different authors vary, this comparative analysis is not rigorous and is only indicative. The objective of this analysis is to provide a general overview of the values of strength and stiffness obtained by other authors for different types of adobe masonry and to contextualise the results obtained in the present study.

There is considerable variability between the results obtained by different authors. In general, however, the results are of the same order of magnitude. The mean compressive strength obtained in the present study is considerably lower than the compressive strength obtained by other authors. The mean shear strength is close to the lower shear strength values obtained by other authors. The stiffness values obtained in this study are markedly greater (on average, 10 times greater) than those obtained by other authors, which is consistent with the tendency observed in the previous study focused on the adobes of Aveiro district, presented in Chapter 4. In this case, however, the difference is significantly lower than that observed in the testing of adobe specimens. The higher stiffness values may be justified by the differences in the materials and construction procedures used. As explained in Chapter 4, the adobes used in Aveiro district were made with sandy soils, which sometimes included some gravel in their composition, and were stabilised with a significant fraction of lime binder, while the adobes used by other authors were not stabilised and were made with finer soils. In the previous study, presented in Chapter 4, it was observed that in at least two of the studies carried out by other authors the deformation of specimens was measured on the loading system and that this leads to higher values of deformation and, consequently, to lower values of modulus of elasticity. In the present study, however, it was observed that, in general, the other authors measured the deformation of adobe masonry specimens directly on the specimens – this may explain why the stiffness values determined by other authors are closer to those obtained in the present study, when compared to what was observed in Chapter 4.

Table 5.3: Results obtained by different authors in simple and diagonal compression tests conducted on adobe walls.

	Reference		Present study	Meli (2005)	S.Bartolomé & Pehovaz (2005)	Torrealva & Acero (2005)	Liberatore et al. (2006)	Yamín et al. (2007)	Wu et al. (2013)
	Location		Portugal	Mexico	Peru	Peru	Italy	Colombia	China
Adobe bricks	Composition		Arenaceous soil and lime binder	Clayey soil	Soil, coarse sand and straw, in proportion 5:1:1	Soil, coarse sand and straw, in proportion 5:1:1	Silty sand	Clayey soil, with or without natural fibres	Soil with 44% clay-silt and 56% sand, and straw (0.5%)
	Condition		Collected from existing constructions	New (produced for the study)	New (produced for the study)	New (produced for the study)	Collected from existing constructions	Collected from existing constructions	New (produced for the study)
	Comp. strength (MPa)		0.47	0.51-1.57	2.94	Not indicated	0.29-1.56	2.84	1.66
	Tensile strength (MPa)		0.14 [a]	0.20-0.43 [b]	Not indicated	Not indicated	0.17-0.40 [b]	0.49 [b]	Not indicated
Simple compression test	No. of specimens		5	Not indicated	4	5	…	15	9
	Dimensions (m) [c]	*H:*	1.26	Not indicated	0.58	0.43	…	Not indicated	0.53
		W:	1.26		0.38	0.25			0.29
		t:	0.36		0.24	0.25			0.20
	Aspect ratio (H / t)		3.6	…	2.4	1.7		…	2.7
	Aspect ratio factor (‘k_a’) [d]		0.91	…	0.83	0.77	…	…	0.84
	Comp. strength (‘f_c’) (MPa)		0.33	1.32	0.86	0.85	…	1.10	0.94
	Unconfined comp. strength (k_a . f_c) (MPa)		0.30	…	0.71	0.65	…	…	0.79
	Modulus of elasticity (MPa)		757	245	Not indicated	432	…	98	34 [e]
Diagonal compression test	No. of specimens		5	Not indicated	4	3	1	10	…
	Dimensions (m) [c]	*H:*	1.26	Not indicated	0.80	0.50	0.90	0.75-1.00	…
		t:	0.36		0.24	0.25	0.20	0.15-0.40	…
	Shear strength (‘f_v’) (MPa)		0.03	0.14	0.11	0.07	0.02	0.03	…
	f_v / f_c (%)		8	10	13	8	…	3	
	f_v / (k_a . f_c) (%)		9	…	15	10	…	…	…
	Modulus of rigidity (MPa)		413	Not indicated	Not indicated	Not indicated	Not indicated	27	…

[a] Splitting tensile strength.
[b] Flexural tensile strength.
[c] H - Height; W - Width; t - Thickness.
[d] Calculated according to ‘NZS 4298’ (SNZ 1998b).
[e] Initial tangent modulus of elasticity.

There is no clear correlation between the compressive strength of the wall panels and the compressive strength of the adobe bricks. In general, however, walls have significantly lower strength values than the respective adobe bricks. Finally, the ratios between the shear strength and the compressive strength of the walls tested by different authors are all of the same order of magnitude and close to the ratio proposed in ‘NZS 4297’ (SNZ 1998a) for ‘special grade earth construction’ (7%).

5.4. Conclusions and final remarks

Ten full-scale adobe wall panels were built in the laboratory and tested in simple compression and diagonal compression, and the results obtained were presented and analysed in this chapter. A summary of the main results is displayed in Table 5.4.

Table 5.4: Summary of the results obtained.

	Compressive strength (kPa)	Strain at peak stress (‰)	Modulus of elasticity (MPa)	Poisson's ratio	Shear strength (kPa)	Shear strain at peak stress (‰)	Modulus of rigidity (MPa)
Mean:	331	3.06	757	0.16	26	0.54	413
CV:	17%	28%	8%	63%	16%	30%	16%

The following key observations and conclusions can be drawn from the results obtained:

- The walls tested show brittle failure, i.e. after maximum stress is reached, failure occurs rapidly, with small deformations;
- The walls subjected to simple compression display a damage pattern consisting predominantly of vertical scattered cracking, with cracks initiating mainly in the interface between adobe bricks and head joint mortar; the damage suffered by walls tested in diagonal compression consists mainly of one or two large central cracks, approximately vertical;
- The mean compressive strength obtained for the walls tested in simple compression is about 70% of the mean compressive strength of the adobes and mortar used in the construction of the walls; the discontinuity caused by vertical joints contributes to the lower compressive strength of the masonry walls;
- The compressive and shear strength values obtained are significantly lower than the strength limits indicated in ‘NTE E.080’ (ICG 2006) for adobe masonry;
- The stiffness of the adobe walls is markedly lower than the stiffness of the adobe bricks; the stiffness of the walls, however, is significantly higher than that obtained by other authors in the testing of adobe masonry representative of adobe construction in other regions of the world (e.g. Meli (2005), Yamín et al. (2007), Wu et al. (2013)),

which is likely justified by the differences in the materials and construction methods used in Aveiro district and in the other regions;

- The influence of the displacement transducers gage length in the results obtained in diagonal compression tests may be significant and thus should be considered in the technical standards that address diagonal compression and in the discussion of results obtained in this type of test; further studies focused on the influence of the use of different gage lengths are recommended.

It was observed that the variability of some of the results obtained is significant. The variability of Poisson’s ratio is particularly high, which must be in part due to the difficulty in measuring low deformation in adobe walls at low levels of compressive stress. This significant variability is in agreement with the variability of results observed in the testing of adobe specimens, reported in Chapters 3 and 4. As explained in these chapters, a large variability of results was expected. The mechanical properties of the adobes used in the construction of the walls show high variability because, traditionally, the materials used in the production of adobes had many heterogeneities, and production procedures could vary significantly, even for the same adobe building. The traditional procedures used in the construction of the walls certainly introduced variations that also contribute to the variability of results. Other authors testing adobe and masonry specimens report similar or even higher variability of results (e.g. Meli (2005), Torrealva and Acero (2005), Liberatore et al. (2006), Yamín et al. (2007)).

In the development of the present study, a lack of technical standards with detailed indications for the testing of adobe masonry was observed. As previously noted in Chapters 3 and 4, it is important to develop standardised procedures adequate to earthen construction, focused not only on new buildings but also on existing ones.

This study is a first contribution to the study of the mechanical properties and behaviour of the adobe masonry traditionally used in Aveiro district. The results obtained can support the development of further studies on the behaviour of adobe structures and can also assist the rehabilitation and strengthening of existing adobe constructions.

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Chapter 6

In-plane cyclic behaviour of a full-scale adobe wall

The work reported in this chapter is presented in: Silveira, D., Varum, H., Costa, A., and Pereira, H. “In-plane cyclic behavior of a full-scale adobe masonry wall.” *Eng. Struct.* (submitted for publication on 28 October 2015).

6.1. Introduction

As previously explained in Chapter 1, adobe construction, if not effectively designed and strengthened, may have a very poor response when subjected to seismic demands. This deficient performance is linked to the low tensile and shear strength and brittle behaviour of adobe masonry (Yamín et al. 2003; ICG 2006). In addition, adobe structures have a very large mass and thus are subjected to high inertial forces during earthquakes (Blondet 2008). There are many examples of recent earthquakes that caused extensive damage to adobe construction, leading to great human and material losses (e.g. JSCE (2001), Mahdi (2005), Blondet (2008), Elnashai et al. (2010)).

Knowledge of the behaviour of adobe structures when subjected to seismic loads is fundamental for the development of effective repair and retrofitting solutions. With the purpose of contributing to this knowledge, an experimental study of the behaviour of a full-scale adobe wall subjected to in-plane horizontal cyclic loading was carried out and is presented in this chapter.

A brief review of previous research focused on the seismic testing of adobe structural systems and development of seismic retrofitting solutions, and a further description of the motivation and summary of the present study are presented below.

6.1.1. Research on the seismic testing and retrofitting of adobe structural systems

Peru is a country that has been subjected to frequent earthquakes along the time and that has a great tradition in construction with adobe (Vargas et al. 2005). In many of these earthquakes, adobe structures performed poorly, causing severe human and material losses (Vargas et al. 2005). For this reason, Peruvian institutions like the National University of Engineering and the Pontifical Catholic University of Peru (PUCP) have developed research with the objectives of studying the structural behaviour of adobe masonry and testing effective repair and seismic retrofitting solutions (e.g. Zavala and Igarashi (2005), Blondet et al. (2011)). PUCP, in particular, has developed important research that started in the early seventies and continues to the present (Blondet et al. 2011).

In a first phase, PUCP conducted static tests using a reinforced concrete tilting platform and a reaction wall (Corazao and Blondet 1973). Later, in 1984, the first seismic tests were conducted: full-scale modules of adobe houses without roofs were tested using a unidirectional shaking table (Vargas et al. 1984). This type of test continued to be developed in this decade, with the objective of studying the influence of materials, retrofitting systems, roofing, and constructive techniques in the seismic behaviour of adobe buildings (Ottazzi et al. 1989). In the nineties, PUCP focused the experimental work on existing adobe constructions. Different retrofitting materials and solutions were tested in U-shaped walls and house modules (Zegarra et al. 1997a; Zegarra et al. 1997b), and the technique that proved to be the most effective, based on the application of a welded steel mesh, was used successfully to reinforce many adobe houses in different parts of Peru (Zegarra et al. 1999). In the beginning of the 21st century, a new line of investigation was launched, with the objective of developing retrofitting systems using industrial materials, more economical and easily found in the market than the materials and solutions previously studied. Cyclic tests were conducted on full-scale double-T shaped walls, and seismic tests were performed on full-scale house models and vaulted structural models

(Blondet et al. 2005; Blondet et al. 2006; Torrealva et al. 2009). In these tests, a retrofitting solution using polymer mesh was evaluated with success. Recently, PUCP has been conducting research with the objective of developing effective solutions to repair cracks in seismically damaged earthen walls. The procedures for mud grout injection have been assessed in dynamic and cyclic tests conducted on full-scale models (Blondet et al. 2014).

Authors from other institutions and countries have also developed relevant research to study the seismic behaviour of adobe constructions and develop effective seismic retrofitting solutions. In the University of the Andes, in Colombia, for example, full-scale adobe walls were tested with in-plane and out-of-plane loading, 1:5 scale house models were tested on a shaking table, and 1:1.5 scale house models were subjected to horizontal cyclic loading, using different retrofitting solutions (Yamín et al. 2003). In the National Autonomous University of Mexico, walls were submitted to cyclic lateral loading, and rural house models (1:2.5 scale), retrofitted with different techniques, were submitted to dynamic tests on a shaking table (Meli 2005). In the University of Technology, in Sydney, Australia, U-shaped adobe walls and a house model (1:2 scale) with various retrofitting solutions were also tested on a shaking table (Dowling and Samali 2009).

In the nineties, the Getty Conservation Institute launched the Getty Seismic Adobe Project, with the objectives of investigating the seismic performance of historic adobe structures and developing effective seismic retrofitting solutions with limited impact on historic buildings. A large part of the investigation was focused on the shake table testing of reduced-scale models of adobe walls and buildings. Small-scale building models (1:5 scale) were tested in the Stanford University, in the USA, and large-scale building models (1:2 scale) were tested in the Institute of Earthquake Engineering and Engineering Seismology, in the Republic of Macedonia (Tolles 2009). More recently, the Getty Conservation Institute, with the collaboration of several other institutions, launched the Seismic Retrofitting Project. With this project it is intended to design and test retrofitting techniques that can be easily implemented – resorting to locally available materials and expertise –, using dynamic and static tests and numerical modelling analyses (Cancino et al. 2012).

6.1.2. Motivation and summary

Aveiro district, in Portugal, is located in a region with moderate seismic hazard (reference peak ground acceleration on rock or firm soil (‘a_{gR}’) of 0.35 m/s^2 and 1.1 m/s^2, for the Type 1 and Type 2 spectra, respectively (CEN 2010)). However, the seismic demand on structures may be considerably amplified for soft foundation soils (CEN 2010), which are very common in this region (Bonito 2008). Moreover, a significant percentage of the existing buildings in Aveiro district are in poor state of conservation (Silva et al. 2010; Silveira et al. 2013a) and do not present adequate seismic retrofitting. Thus, in case of an earthquake, these buildings may perform very poorly, which can lead to significant losses.

As presented above, important research on the seismic behaviour of adobe structures and development of seismic retrofitting solutions has been conducted in the last decades. More research, however, is necessary for the enrichment of the existing knowledge concerning the seismic behaviour of adobe structures. The study of the cyclic behaviour of adobe masonry with different characteristics, i.e. built with different techniques and materials, is essential for the development of effective retrofitting solutions adapted to the different adobe masonry systems used throughout the world. In particular, the structures made with lime adobe, which are common in Portugal and in other regions of the world (e.g. Dipasquale and Mecca (2011), Lopez et al. (2014)), are in need of special research attention.

To contribute to the existing knowledge, research focused on the seismic behaviour of the adobe constructions of Aveiro district was carried out. The main objective of this research was to contribute to the understanding of the behaviour of adobe structures when subjected to in-plane horizontal cyclic demands. To this end, a full-scale adobe wall with double-T shape was built in the laboratory and subjected to in-plane horizontal cyclic loading of increasing amplitude. The wall was built with adobes taken from an existing construction and mortar formulated with composition similar to that traditionally used. With the test carried out, it was possible to assess the behaviour of the wall in terms of: shear stress versus horizontal drift and moment versus rotation relationships; maximum lateral strength; drift and rotation at peak stress; evolution of stiffness, lateral displacement, dissipated energy, and natural frequency; and damage pattern. A comparison of the results obtained in this study with the results obtained by other authors in in-plane cyclic or monotonic tests carried out on simple or double-T shaped walls, representative of adobe construction in different countries, was also performed. This experimental work assisted the subsequent development of an effective repair and retrofitting solution (Figueiredo et al. 2013) and can also support other studies, such as the development of numerical models to simulate the seismic response of adobe structures.

6.2. Wall characteristics

6.2.1. Geometry

A full-scale adobe wall was built in the laboratory with the shape of a double-T in plan view (Figure 6.1). With this shape it was intended to simulate the influence of the connection with adjacent orthogonal walls, typical in existing constructions. The wall was built with a height of 3.07 m and mean thickness of 0.29 m. The main longitudinal wall had a length of 3.50 m and the two transverse walls were 1.70 m long.

Figure 6.1: Construction of the full-scale adobe wall.

6.2.2. Materials

Adobes

The adobes used in the construction of the wall were collected from a representative adobe house that was undergoing demolition (house '*H*20', as defined in Chapter 4), located in the parish of Cacia, in Aveiro municipality, and were in a good state of conservation (Figure 6.1a). The adobes were made with arenaceous soil and air-lime binder, as is traditional in Aveiro district. The mechanical properties of ten adobe bricks taken from this building were assessed in Chapter 4 (Silveira et al. 2013b). The adobes had the following characteristics (mean values): dimensions of 0.44 x 0.24 x 0.12 m^3; specific weight of 15 kN/m^3 (CV = 6%); compressive strength of 0.46 MPa (CV = 21%); modulus of elasticity of 17742 MPa (CV = 36%); and splitting tensile strength of 0.14 MPa (CV = 21%).

Mortar

The mortar used for the joints and render of the wall (Figure 6.1d) was formulated with a composition similar to that traditionally used in the adobe constructions of Aveiro region. A hydrated lime: 'earth' ratio of 1:3, in terms of bulk volume, was adopted. The 'earth' used consisted of a mixture of slightly clayey soil and sand (in a ratio of 1:2). The soil was taken from a site referenced by former mortar and adobe manufacturers. The earth mixture was classified as sand (ISO 2004), including small fractions of gravel (particles with size between 2 mm and 9.5 mm), clay, and silt (particles with size lower than 0.075 mm). The mortar used had the following mean characteristics: specific weight of 18 kN/m^3; compressive strength of 0.67 MPa (CV = 8%); and flexural tensile strength of 0.23 MPa (CV = 27%). The compressive and flexural strength of the mortar were determined according to the indications of 'EN 1015-11' (CEN 1999). The modulus of elasticity of the mortar was not determined. This modulus, however, is likely close to the modulus of elasticity of the adobes used, given that the composition, production, and curing procedures used for the two materials are similar. After being rendered with lime mortar (Figure 6.1e), the wall was painted with lime paint (Figure 6.1f), as traditionally done, to facilitate the identification of cracks during testing.

6.2.3. Foundation

The foundation of the wall consisted of a reinforced concrete pad footing fixed to the reaction floor of the laboratory with prestressed threaded rods (Figure 6.1a). The first adobe layer of the wall was connected to the foundation concrete block with cement mortar to prevent sliding failure at the base during the cyclic test (Figure 6.1a).

6.3. Testing

The wall was tested approximately 50 days after construction. A horizontal cyclic force of increasing amplitude was applied at a height of approximately 2.60 m from the base of the wall, until failure. The force was applied with a hydraulic actuator with a maximum load capacity of 100 kN (Figure 6.2a). A transversal steel beam was mounted to transfer the horizontal load from the hydraulic actuator to the wall (Figure 6.2a). Two horizontal prestressing steel rods were placed longitudinally, one at each side of the wall, connecting the load application steel beam to a similar steel beam on the opposite transverse wall, to allow for the application of loading in both senses (Figure 6.2b). To simulate the dead and quasi-permanent live loads typical in adobe buildings, a vertical uniformly distributed load with a total value of 20 kN (19.7 kPa – corresponding to 6% of the compressive strength of adobe masonry as determined experimentally in Chapter 5 (Silveira et al. 2015)) was added to the top of the wall (Figure 6.2b). In the calculation of this vertical load the following elements were considered: 0.40 m high vertical extension of the wall; false ceiling (0.50 kPa); and timber roof with ceramic tiles (0.88 kPa). This vertical load consisted of steel beams placed on the top of the wall and did not add any restrictions to the boundary conditions of the wall. The general scheme of the test set-up is presented in Figure 6.3.

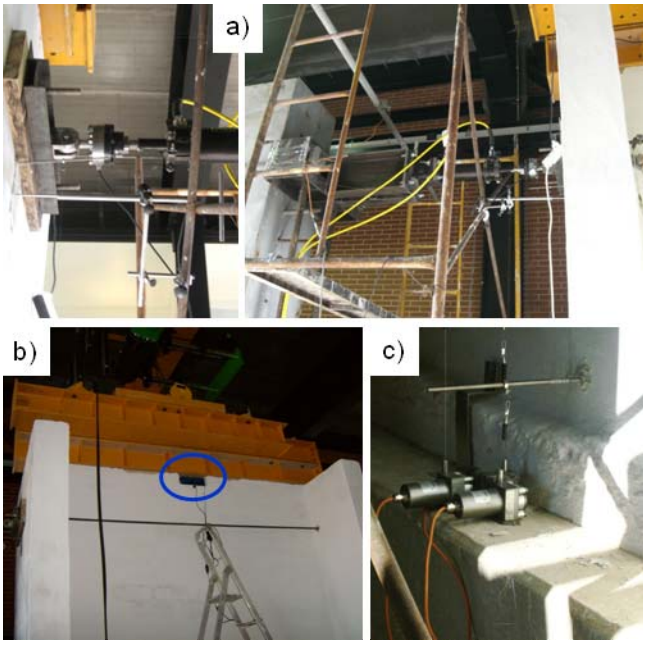

Figure 6.2: Test set-up and instrumentation: a) hydraulic actuator; b) additional vertical load, seismograph, longitudinal steel bar; c) displacement transducers.

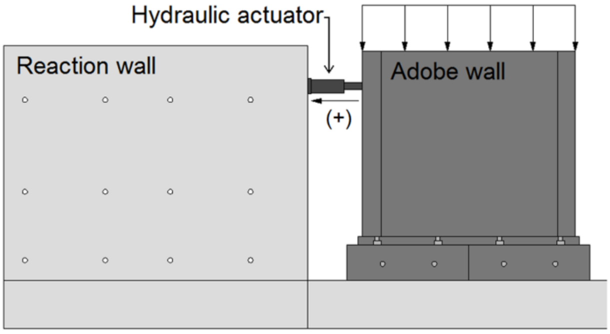

Figure 6.3: General scheme of the test set-up (adapted from Pereira (2008)).

The evolution of the deformation of the wall during the test was measured with displacement transducers (electronic potentiometers) placed at points representative of the structural response of the wall (Figures 6.2c and 6.4). The test cycles, in terms of the maximum horizontal drift imposed, are presented in Table 6.1. The horizontal drift was calculated by dividing the horizontal displacement of the wall, measured at the level of force application, by the height of force application.

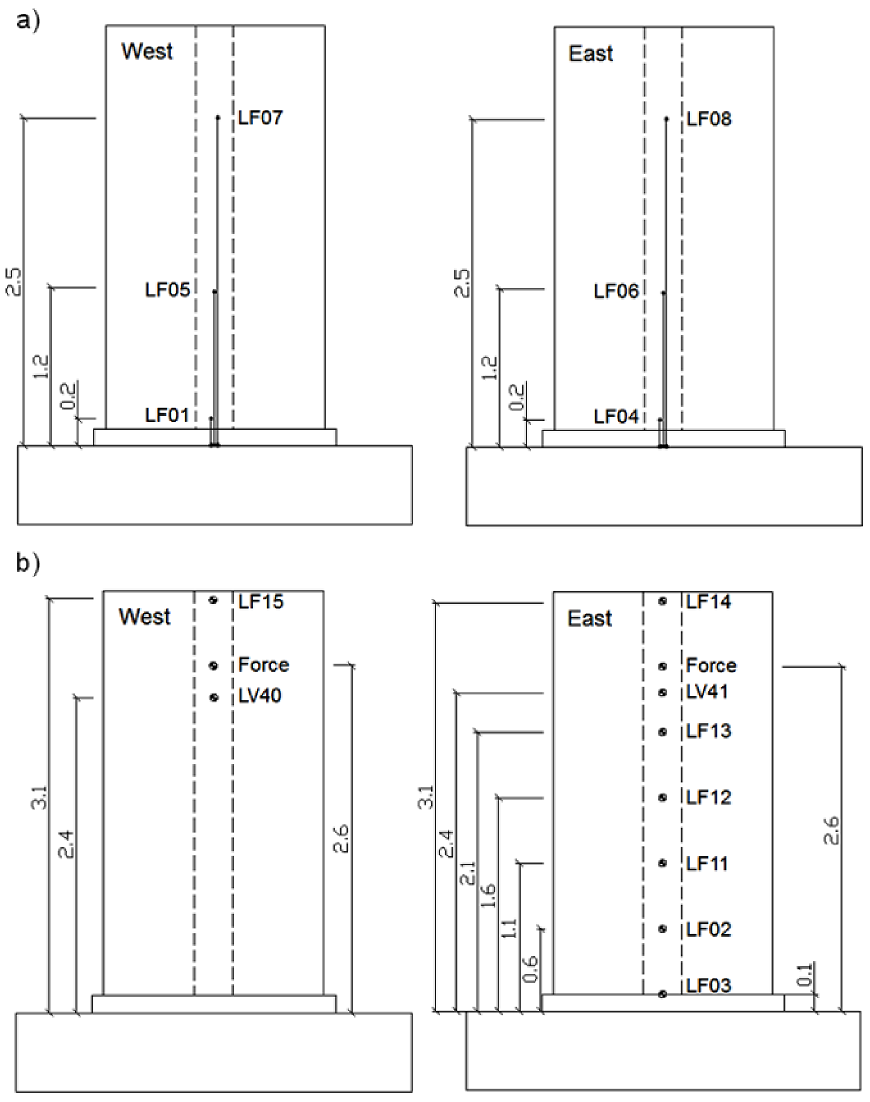

Figure 6.4: a) Vertical and b) horizontal displacement transducers.

Table 6.1: Test cycles, in terms of maximum horizontal drift.

Test cycles					
Maximum drift:	0.01%	0.02%	0.03%	0.05%	• 0.03% at peak stress • 0.16% at 80% of peak stress
No. of repetitions:	2	3	1	1/2	1/2

A seismograph was placed on the top of the wall (Figure 6.2b), with the objective of estimating the evolution of the first natural frequency of the wall during the test. This seismograph measured the acceleration caused by an excitation induced on the wall before the cyclic test and between loading cycles.

6.4. Results

6.4.1. Stress-drift and moment-rotation relationships

The shear stress versus horizontal drift curve obtained from the cyclic test is presented in Figure 6.5. The shear stress was determined at the base of the wall by dividing the lateral force by the cross sectional area of the wall, considering only the contribution of the wall's web. The curve is represented until the point where the shear stress decreases to about 80% of its maximum value. For this point, the corresponding drift is of 0.16%.

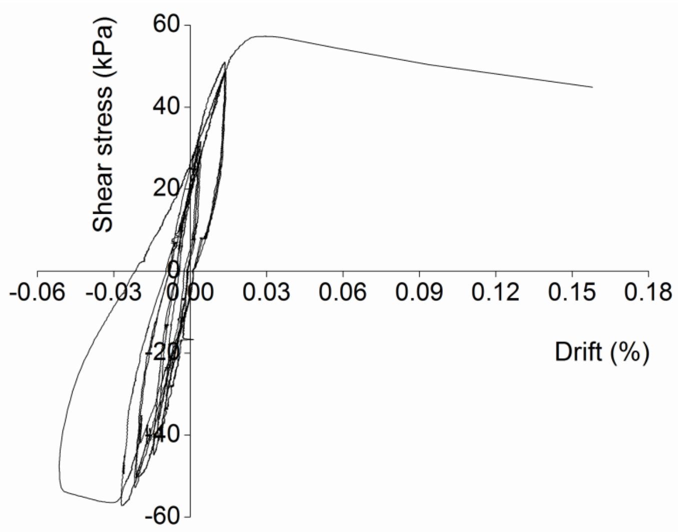

Figure 6.5: Cyclic response of the adobe wall in terms of shear stress versus horizontal drift.

A maximum lateral force of 58.1 kN, with a corresponding maximum shear stress of 57.3 kPa, was reached for a drift of 0.03%. The maximum lateral force corresponds to 56% of the total vertical dead load. The initial tangent shear stiffness, calculated as the slope of the stress-drift curve in the initial linear region, is of 738 MPa. The secant shear stiffness at maximum stress is of 192 MPa. Throughout the test, and until the last cycle, the response of the wall is almost linear. During the last cycle, however, the wall displays evident non-linear incursions.

The curves moment at the base of the wall versus rotation measured at the base of the wall and moment at the base of the wall versus rotation measured at 2.50 m high (i.e. close to the level of force application) are presented in Figure 6.6. The moment was calculated by multiplying the lateral force by the height of force application. The curves are represented until a decrease in moment of approximately 20% relatively to its maximum value.

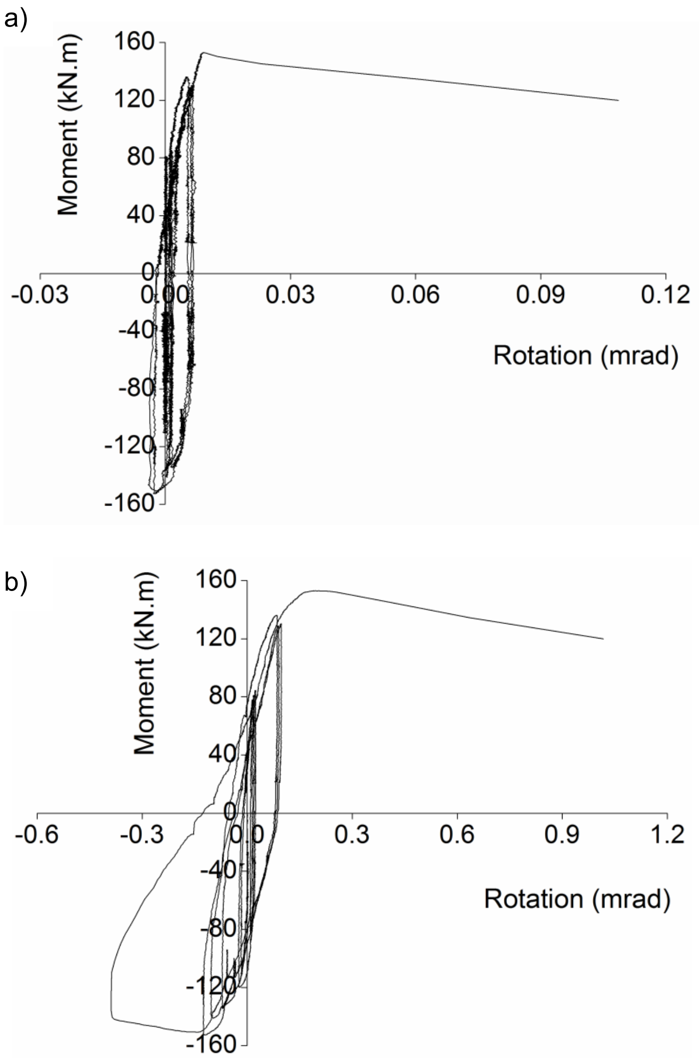

Figure 6.6: Cyclic response of the adobe wall in terms of moment versus rotation: a) at the base; b) at 2.50 m high.

A maximum moment of 153 kNm, with a corresponding rotation at the base of 0.009 mrad and at 2.50 m high of 0.205 mrad, were observed. The curve moment versus rotation at 2.50 m high (Figure 6.6b) presents a path similar to that of the shear stress versus drift curve (Figure 6.5). In the moment versus rotation at the base curve (Figure 6.6a), the wall shows an almost linear response throughout the test until failure. In the moment versus rotation at 2.50 m high curve (Figure 6.6b), the same quasi-linear behaviour is observed until the beginning of the last test cycle, and in the last cycle a pronounced non-linear behaviour is displayed.

As can be observed in all three curves (Figures 6.5 and 6.6), the wall presents a quasi-elastic response, in terms of drift and rotation, and brittle behaviour (i.e. failure occurs suddenly, for small values of drift and rotation).

6.4.2. Comparison of the shear strength with the design seismic action

The design seismic action for Aveiro was calculated according to 'Eurocode 8' (CEN 2010), considering: i) a building of importance class II ('ordinary building'); ii) soil of type D ('deposits of loose-to-medium cohesionless soil – with or without some soft cohesive layers – or of predominantly soft-to-firm cohesive soil'); iii) period of 0.04 s (based on the initial frequency measured, as will be presented in subsection 6.4.5); iv) and behaviour factor ('*q*') of 1 (taking into account the brittle behaviour of the adobe wall). For these conditions, the most severe scenario corresponds to the seismic action of Type 2. A design seismic action corresponding to 31% of the total vertical load was obtained. The in-plane strength of the wall, which corresponds to 56% of the total vertical load, is thus greater than the design seismic action.

It is important to note that this analysis has obvious limitations, since the structure tested corresponds to a small part of a building and the force considered is acting only in the plain of the wall. In addition, the test is not dynamic, but quasi-static, which also limits the analysis. This analysis was presented here simply in an attempt to contextualise the strength value obtained in light of a concrete seismic scenario.

6.4.3. Lateral displacement profile

The evolution of the lateral displacement profile of the wall is presented in Figure 6.7. The displacement profile evidences an approximately linear response, combining shear and flexural response components up to the cycle that corresponds to a maximum drift of 0.03%. From the analysis of the shape of the displacement profile for the half cycle that corresponds to a maximum drift of 0.05%, it can be observed that cracking occurred at a height between 0.5 m and 1.0 m. It can also be concluded that no sliding occurred at the base of the wall, since the horizontal displacement transducers located close to the base display negligible displacements.

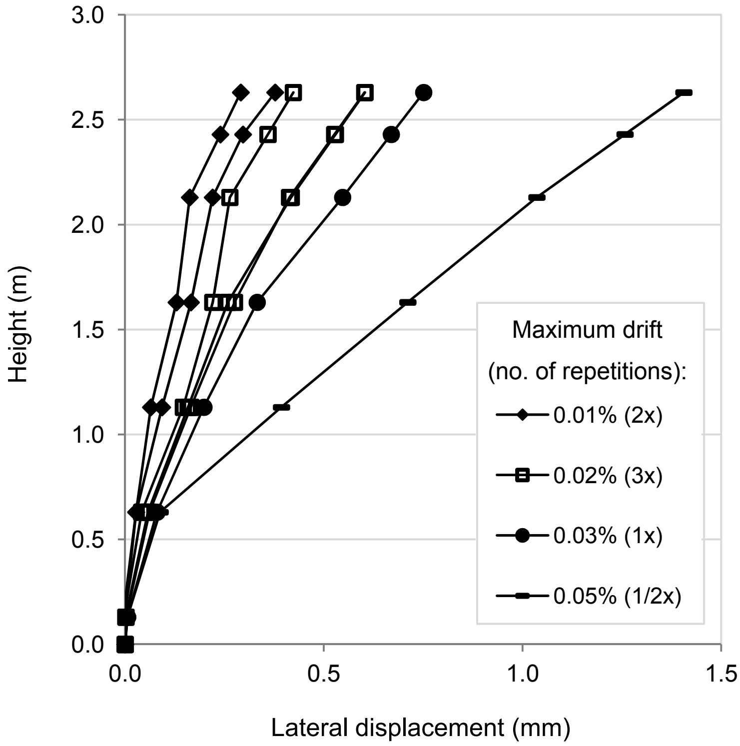

Figure 6.7: Evolution of the lateral displacement profile.

6.4.4. Dissipated energy

The evolution of the total energy (i.e. the sum of the stored potential energy, released elastic energy, and dissipated plastic energy) of the wall during the cyclic test was determined by integration of the force versus displacement curve until the end of the first half of the last cycle (maximum drift of 0.05%) and is presented in Figure 6.8. A graph with the values of dissipated energy per test cycle is presented in Figure 6.9.

From the analysis of the evolution of the dissipated energy, it can be observed that:

- In the first test cycles (up to a drift of 0.01%), the wall suffers almost no damage, which is reflected in low values of energy dissipation;

- In the following cycles (maximum drifts of 0.02% and 0.03%), the energy dissipation increases steadily as the wall begins to display tenuous damage that propagates slowly;
- In the first half of the last cycle (maximum drift of 0.05%), there is a higher increase in energy dissipation, as this half cycle corresponds to the onset of the first large diagonal crack, even though at this point the crack opening is small.

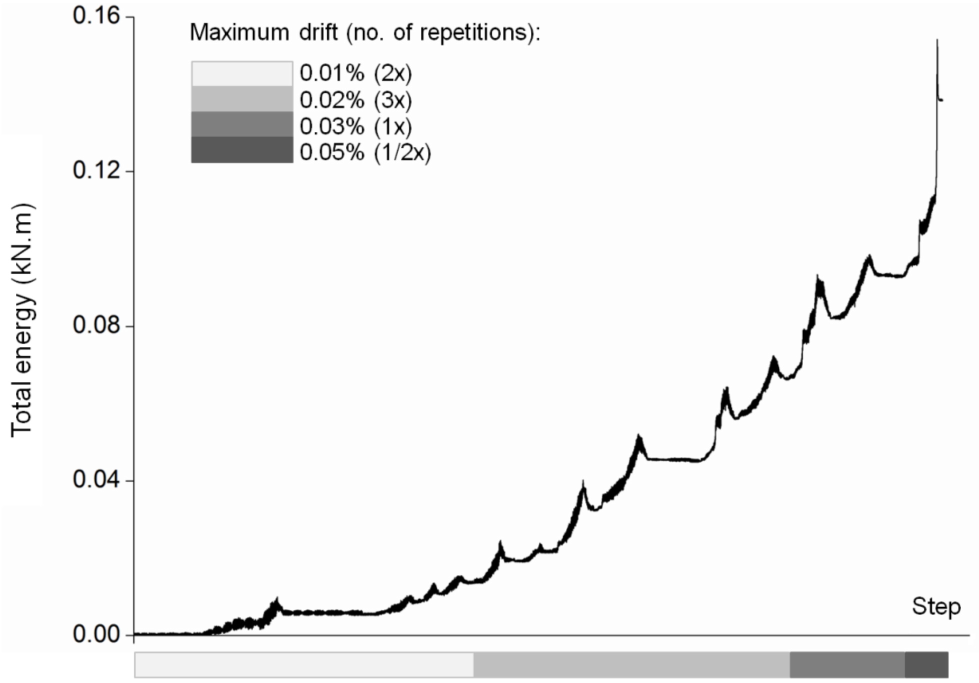

Figure 6.8: Total energy evolution.

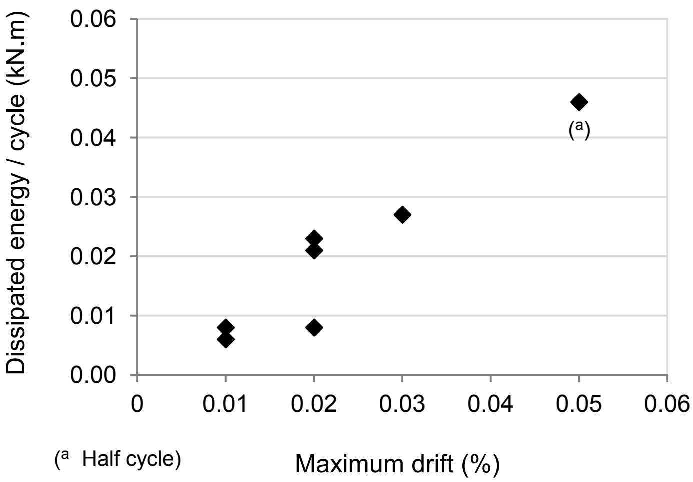

Figure 6.9: Dissipated energy per test cycle.

6.4.5. First natural frequency

The evolution of the first longitudinal natural frequency of the wall throughout the cyclic test, measured in the intervals between test cycles, is presented in Figure 6.10. In the first cycles, the natural frequency of the wall decreases smoothly. After the maximum capacity of the wall is reached, the decrease in frequency is more accentuated, corresponding to a severe increase in the damage of the wall. At the end of the cyclic test, the natural frequency of the wall decreased by 21% when compared to the value of the original wall.

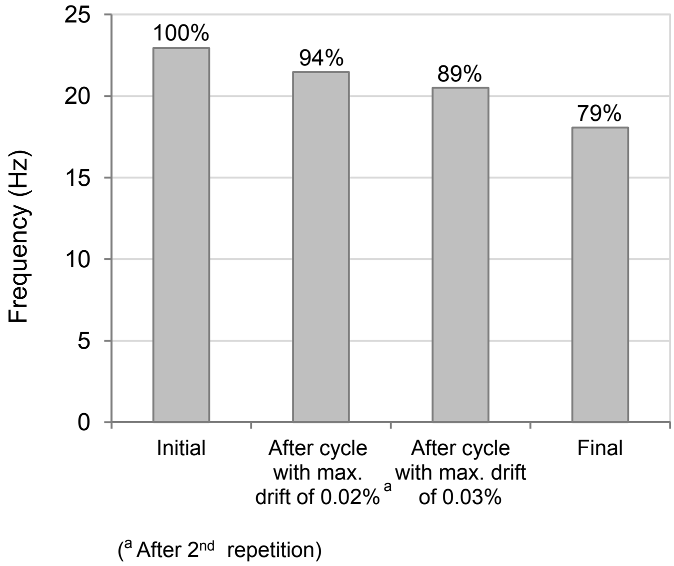

Figure 6.10: Evolution of the first longitudinal natural frequency.

6.4.6. Damage evolution and pattern

Until the last test cycle, the damage observed on the wall is light. The first half of the last cycle (maximum drift of 0.05%) corresponds to the formation of the first large diagonal crack. At this phase of the test, however, this crack is hardly visible. In the last half cycle of the test, which corresponds to the onset of failure, another large diagonal crack is formed abruptly in the opposite direction. After this last cycle, the test was extended during another half cycle in the opposite direction, increasing the opening of the first diagonal crack and thus leading to the formation of the X-shaped pattern that is typical in masonry walls subjected to in-plane cyclic loads (Figure 6.11), such as those induced by earthquakes.

Figure 6.11: Damage suffered by the wall.

The cracks in the wall follow along mortar joints in a large part of their path, creating a stepped pattern. Joints can thus be areas of weakness in adobe masonry subjected to in-plane cyclic loads. This is consistent with the damage observed in the adobe wall panels tested in simple and diagonal compression, presented in Chapter 5, in which cracks were generally initiated in the interface between mortar and adobe bricks. The strength of the adobe bricks used in the adobe masonry of Aveiro district is normally close to the strength of the mortar, and thus cracks initiating in mortar joints are generally a result of insufficient bond strength between the two materials (Silveira et al. 2015).

6.4.7. Comparison with the results obtained by other authors

A summary of the results obtained by other authors (Blondet et al. 2005; Meli 2005; Zavala and Igarashi 2005; Yamín et al. 2007) in in-plane cyclic or monotonic tests conducted on simple adobe walls or double-T shaped adobe walls, representative of adobe construction in different countries, is presented in Table 6.2, together with a summary of the results obtained in the present work. Some of the values displayed in Table 6.2 were estimated from the graphs presented in the literature sources and some of the values result from a recalculation of the results reported in these sources, in order to facilitate the comparison with the results of the present work. The test procedures and characteristics of the specimens used by the different authors vary, and thus this analysis is not rigorous and is only indicative. With this analysis it is intended to provide a general overview of the results obtained by other authors and to assess the global differences, in terms of performance, between adobe walls with different characteristics.

From the analysis of the data presented in Table 6.2, it can be concluded that there is significant variability between the results obtained by different authors. Nevertheless, results are consistent in terms of order of magnitude, with the exception of the stiffness values obtained in the present work, which are significantly higher (on average, 12 times higher) than those obtained by other authors. This is consistent with the tendency observed in the previous experimental studies focused on adobe specimens and adobe wall panels of Aveiro district, presented in Chapters 4 and 5. The different materials and construction techniques used may justify the differences found – as explained in Chapters 4 and 5, the adobes traditionally used in Aveiro district were made with sandy soils that sometimes included some gravel in their composition and were stabilised with lime binder, while the adobes used by other authors were made with finer soils and were not stabilised.

From the analysis of Table 6.2, it can also be observed that there is no apparent correlation between the shear strength of the walls and the compressive strength of the adobes. However, since the procedures adopted in the testing of adobe specimens may strongly influence the compressive strength obtained (Silveira et al. 2013b), the values of compressive strength obtained by different authors may not be directly comparable.

Finally, it can be noted that diagonal cracks occurred in all the walls tested, which is typical in masonry walls subjected to in-plane cyclic loading. In most cases, cracks follow along mortar joints in a large part of their path, forming a stepped pattern.

Table 6.2: Results obtained by different authors in in-plane cyclic or monotonic tests conducted on adobe walls.

	Reference	Present study	Blondet et al. (2005)	Meli (2005)	Zavala & Igarashi (2005)	Yamín et al. (2007)
	Location	Portugal	Peru	Mexico	Peru	Colombia
	Type of test	In-plane cyclic test	In-plane cyclic test	In-plane cyclic test	In-plane monotonic test	In-plane cyclic test
	No. of specimens	1	1	2	4	3
Dimensions	Height (m)	3.07	1.93	2.00	2.30	2.00
Dimensions	Length of main longitudinal wall (m)	3.50	3.06 [a]	2.00	2.45	2.50
Dimensions	Length of transverse walls (m)	1.70	2.48	…	…	…
Dimensions	Thickness (m)	0.29	0.30	Not indicated	0.20	0.40
Adobe bricks	Composition	Arenaceous soil and lime binder	Soil, coarse sand, and straw in proportion 5:1:1	Clayey soil	10-20% clay, 15-25% silt and 55-70% sand [b]; addition of straw or *ichu*	Clayey soil, with or without natural fibres
Adobe bricks	Condition	Collected from existing constructions	New (produced for the study)	New (produced for the study)	New (produced for the study)	Collected from existing constructions
Adobe bricks	Compressive strength (MPa)	0.46	Not indicated	0.51-1.57	≥ 1.18	2.84
Adobe bricks	Tensile strength (MPa)	0.14 [c]	Not indicated	0.20-0.43 [d]	Not indicated	0.49 [d]
	Additional vertical load	20 kN	Concrete beam placed on top of the wall	Not indicated	Not indicated	3 levels: 20, 50 and 70 kN
	Maximum shear stress (kPa)	57	41	98	23	12 [e]
	Drift at peak stress (%)	0.03	0.10	Not indicated	0.17	0.07 [e]
	Initial tangent stiffness (MPa)	738	86	160	70	29 [e, f]
	Secant stiffness at peak stress (MPa) (percentage of initial stiffness)	192 (26%)	41 (48%)	Not indicated	14 (20%)	17 [e] (60%)
Damage pattern	Orientation	Diagonal tendency	Diagonal tendency	Diagonal tendency	Diagonal tendency	Diagonal tendency
Damage pattern	Path	Generally following along mortar joints	Generally following along mortar joints [g]	Not indicated	Following along mortar joints	Generally following along mortar joints

[a] The main longitudinal wall included a central window opening.
[b] Produced according to 'NTE E.080' (ICG 2006).
[c] Splitting tensile strength.
[d] Flexural tensile strength.
[e] Considering the additional vertical load of 20 kN.
[f] Calculated considering linearity until the onset of cracking.
[g] With special concentration around the central window opening.

6.5. Conclusions and final remarks

A double-T shaped adobe wall was constructed in the laboratory and subjected to in-plane cyclic loading until failure, and the results obtained were presented and analysed in this chapter. The following key observations and conclusions can be drawn from the results obtained:

- The wall presents brittle behaviour, i.e. suffers small drift and rotation until sudden failure occurs;
- During the test and until the last cycle, the wall displays an almost linear behaviour; in the last cycle, which corresponds to the onset of failure, the wall presents a pronounced non-linear behaviour;
- The wall has an in-plane strength corresponding to 56% of the total vertical load and an horizontal drift at peak stress of 0.03%; it is important to note that the out-of-plane strength of masonry walls tends to be significantly lower than the in-plane strength, and thus the out-of-plane strength should also be assessed in future research;
- The wall suffers diagonal cracking with an X-shaped pattern – typical in masonry walls subjected to in-plane seismic loads – and cracks generally follow along mortar joints; no sliding occurred at the base of the wall;
- The stiffness of the wall is significantly higher than that of adobe walls tested by other authors in other regions of the world (Blondet et al. 2005; Meli 2005; Zavala and Igarashi 2005; Yamín et al. 2007); this is likely explained by the differences in the materials and construction techniques used.

The experimental work developed and presented in this chapter aims to contribute to strengthen the understanding of the seismic behaviour of adobe masonry structures, particularly of the adobe structures traditionally built in Aveiro district. This research served as the basis for the development and testing of an effective repair and retrofitting solution (Figueiredo et al. 2013) and can also support the development of additional studies on the seismic behaviour of adobe structures.

6.6. References

B

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Chapter 7

Conclusions and future work

7.1. Conclusions

This thesis deals with the constructive and mechanical characterisation of the adobe masonry walls of existing buildings, focusing in particular on the adobe buildings located in Aveiro district, in Portugal. The research includes an inspection of the adobe facade walls of existing buildings and the development of experimental laboratory tests on adobe specimens, adobe wall panels, and a double-T shaped adobe wall.

With the visual and dimensional inspection of the facade walls of twenty-one adobe buildings, presented in Chapter 2, it was possible to characterise the construction details of the facade walls and identify vulnerabilities that may contribute to the instability or poor performance of these structural elements. It was also possible to analyse common defects in adobe facade walls and identify possible causes of these defects. It was observed that many of the existing defects are linked to a lack of regular maintenance measures, deficiencies in existing systems, and also to the use of inappropriate materials in recent interventions. Overall, it was observed that the facade walls of a large percentage of buildings have defects and vulnerabilities that compromise their good performance and, in some cases, the structural integrity of the buildings. It was concluded that the rehabilitation and strengthening of these structural elements, addressing and correcting the causes of existing defects, as well as the regular performance of maintenance interventions, are fundamental.

The experimental study of the mechanical properties and behaviour of adobe bricks from existing constructions was carried out in two phases and presented in Chapters 3 and 4. With this study, it was possible to evaluate the compressive and tensile strength, stress-strain relationships, modulus of elasticity, and Poisson’s ratio of the material, and to assess correlations between different mechanical properties. It was found that the strength values of the adobe specimens are, in general, lower than the limits indicated in existing technical standards for earthen construction (SNZ 1998; ICG 2006; RLD 2009) but are within the range of values obtained by other authors for adobes used in different regions of the world (e.g. Meli (2005), Liberatore et al. (2006), Baglioni et al. (2010)). In the experimental work carried out in the second phase of the study, presented in Chapter 4, it was also possible to create an initial proposal for the correlations between results obtained with different testing procedures. It was concluded that flexural testing of adobe bricks can significantly overestimate tensile strength and that the compressive strength obtained by testing adobe cubic specimens can be very close to that obtained by testing cylindrical specimens (with height to diameter ratio of 2). It was also observed that measuring deformations directly on the test specimens (i.e. using the more appropriate procedure) may lead to values of modulus of elasticity that are significantly higher than those obtained when measurement is performed on the load application system.

The simple and diagonal compression tests performed on ten full-scale adobe wall panels built with adobes taken from an existing building, presented in Chapter 5, allowed the evaluation of the compressive and shear strength, stress-strain relationships, stiffness, Poisson’s ratio, damage pattern, and correlations among different mechanical properties of the adobe panels. It was observed that the adobe walls exhibit brittle behaviour, as expected for this type of material. It was found that the compressive strength of the walls is significantly lower than that of the adobes and mortar, which must be mainly due to the discontinuity created in the interface between adobes and head joint mortar. It was noted that the values of compressive and shear strength are considerably lower than the limits indicated in ‘NTE E.080’ (ICG 2006) and the values of stiffness are significantly greater than those determined by other authors for adobe masonry used in other countries (e.g. Meli (2005), Yamín et al. (2007), Wu et al. (2013)) – observations which are consistent with the tendency observed in the experimental study of adobe bricks, presented in Chapters 3 and 4. It was also concluded that the influence of the displacement transducers gage length in the results of diagonal compression tests can be significant and thus should be taken into account in technical standards that address this type of testing.

In general, the results obtained in the tests performed on adobe specimens and adobe wall panels show significant variability. This can be explained by the fact that the materials and methods of production and construction traditionally used could vary greatly, even within the same construction. Other authors report similar or even higher variability of results in mechanical tests performed on adobe and masonry specimens (e.g. Meli (2005), Torrealva and Acero (2005), Liberatore et al. (2006), Yamín et al. (2007)). It was also observed that the strength values obtained are, in general, lower than the limits indicated in technical standards for earthen construction. This was expected, since existing earthen construction standards (e.g. SNZ (1998), ICG (2006), RLD (2009)) address new buildings, while the present study focuses on materials from constructions that were built until the middle of the 20th century. Existing adobe masonry, in general, cannot meet standard requirements that were developed specifically for new adobe constructions.

The in-plane horizontal cyclic test conducted on a double-T shaped full-scale adobe wall built with adobes collected from an existing building, presented in Chapter 6, allowed the characterisation of the behaviour of the wall in terms of: shear strength; stress-drift and moment-rotation relationships; evolution of stiffness, lateral displacement, dissipated energy, and natural frequency; and damage pattern. The wall exhibited brittle behaviour, with X-shaped cracking, as is typical in adobe walls subjected to in-plane seismic demands. It was noted that the shear strength obtained is consistent with the strength values determined by other authors for adobe walls representative of existing construction in different countries (e.g. Blondet et al. (2005), Meli (2005), Yamín et al. (2007)). It was also noted, however, that the stiffness values are significantly higher than those obtained by other authors, which is in agreement with the tendency observed in the study of adobe specimens and adobe wall panels, addressed in Chapters 3, 4 and 5.

During the preparation of the experimental tests, a lack of comprehensive European and international standards for earthen construction and recommendations in the Eurocodes for this type of construction were noted. Thus, a need for standard procedures and recommendations for earthen construction and, in particular, adobe construction, considering not only new building processes but also existing constructions, is recognised.

More work is needed to validate and further develop the results of the research presented in this thesis. The results and conclusions obtained, nevertheless, are a relevant initial contribution to the enrichment of the knowledge about the traditional adobe building systems, mechanical properties and behaviour of adobe and adobe masonry, and seismic behaviour of adobe structures. The results obtained can be used in future studies on adobe construction – including, for example, in the calibration of numerical models to simulate the behaviour of these structures – and in the development and validation of repair and retrofitting solutions. The procedures adopted in this research and the results obtained can also be taken into account in the development and adaptation of standard recommendations for earthen construction. Overall, this research contributes to support the conservation, rehabilitation, and strengthening of existing adobe buildings in Portugal and other regions of the world and can also assist the design of new adobe constructions. Even though the results and conclusions obtained are only directly applicable to the traditional lime adobe masonry of Aveiro district, the test procedures, types of analyses conducted, and general conclusions are transferrable to the study of other types of adobe masonry.

7.2. Future work

The research presented in this thesis provides a relevant contribution to the knowledge regarding adobe construction. However, more work on this topic is needed to better understand the properties and behaviour of this type of construction and to support the creation of effective rehabilitation and strengthening solutions for existing buildings. The following future developments are thus suggested:

- Continuation of the study of the construction systems and defects of existing adobe buildings, by applying the set of inspection checklists adapted in the present study for adobe construction to a larger number of buildings in Aveiro district as well as in other regions; this study could include a more detailed characterisation of the foundation system of adobe buildings − gathering information about the materials used, dimensions, defects, and state of conservation –, which could be accomplished, for example, by accompanying ongoing rehabilitation interventions in existing buildings;

- Study of the seismic vulnerability of existing adobe buildings in selected representative areas of the world (as initiated by Ferreira (2008) in Aveiro district);
- Development of additional tests for the mechanical characterisation of adobe bricks, with measurement of deformations conducted directly on the test specimens, in order to validate and extend the existing knowledge − particularly concerning the modulus of elasticity and Possion's ratio of the material − and to validate the correlations between the results obtained with different testing procedures;
- Development of additional mechanical tests on adobe wall panels, representative of traditional adobe masonry, to extend the existing knowledge and evaluate other mechanical properties of adobe masonry (such as the flexural, shear, and bond strength); in these tests, the following analyses could be performed: assessment of the mechanical properties of adobe masonry when saturated with water (as initiated by Martins (2015)), and further analysis of the influence of the use of different gage lengths in the results obtained in diagonal compression tests; it is important to note that other mechanical tests on adobe masonry were conducted recently by other researchers at the University of Aveiro, but the results obtained are not yet consolidated;
- Development of cyclic and shaking table tests on adobe masonry structures with different geometric configurations and construction details, representative of existing constructions, to expand the current knowledge; these tests could also be used to assess different seismic retrofitting solutions, including solutions using natural and sustainable materials; it is important to note that cyclic tests on an adobe house model were performed recently by other researchers at the University of Aveiro, but the results are not yet fully consolidated;
- Development of numerical models to simulate the structural behaviour of adobe constructions; the data gathered in the present study can be used to calibrate these numerical models;
- Development of maintenance and rehabilitation solutions and guidelines for existing adobe buildings; research has been carried out recently for this purpose at the

University of Aveiro (e.g. Silva (2012), Tavares et al. (2014), Velosa and Varum (2014), Andrejkovičová et al. (2015)), but further effort in this direction is needed;

- Implementation of activities aimed at transferring the knowledge gained in the research and studies focused on adobe construction to the technicians and entities that manage and intervene in existing adobe buildings;
- Creation of a comprehensive online database, where the existing information regarding adobe construction can be regularly and easily uploaded, organised, and synthesised;
- Development of standard procedures and recommendations for earthen construction – and, in particular, adobe construction – addressing new construction processes and also the rehabilitation of existing constructions; the procedures and results of this and other studies focused on earthen construction can be taken into consideration in this process.

7.3. References

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Appendix A

‘Level 1’ questionnaire (examples)

universidade de aveiro CHARACTERISATION OF ADOBE CONSTRUCTION IN AVEIRO DISTRICT

City Council Questionnaire ADOBE CONSTRUCTION IN ESTARREJA MUNICIPALITY

Parishes	Adobe construction: Very abundant	Moderately abundant / scarce	Nonexistent	No information available
Avanca		X		
Beduído		X		
Canelas		X		
Fermelã		X		
Pardilhó	X			
Salreu		X		
Veiros	X			

Observations:

All parishes have incidence of adobe construction. However, those closer to the Ria, perhaps due to the ease of transportation by boat and the absence of materials other than wood (the soil is sedimentary–sandy) have more adobe construction, made with sand from Esgueira or with mud from the Ria.

There is even a popular aphorism with the following meaning: "If you do not want to leave houses to heirs, build with adobe from the Ria".

Many of these adobes were made in Quinta do Areal, in Esgueira, by Francisco António de Pinho Júnior, and later by his son-in-law Manuel Duarte dos Santos, until the seventies.

(Portuguese to English translation by D. Silveira)

Information by:

Name(s): Arq. José Moutinho

Contact(s):

universidade de aveiro CHARACTERISATION OF ADOBE CONSTRUCTION IN AVEIRO DISTRICT

City Council Questionnaire ADOBE CONSTRUCTION IN VAGOS MUNICIPALITY

	Adobe construction			
Parishes	**Very abundant**	**Moderately abundant / scarce**	**Nonexistent**	**No information available**
Calvão	X			
Covão do Lobo	X			
Fonte de Angeão	X			
Gafanha da Boa Hora	X			
Ouca	X			
Ponte de Vagos	X			
Santa Catarina	X			
Santo André de Vagos	X			
Santo António de Vagos	X			
Sosa	X			
Vagos	X			

Observations:

In the municipality area there are many buildings in which the presence of adobe is important. I may advance, without having quantitative information, that in all parishes there are many adobe masonry buildings.

(Portuguese to English translation by D. Silveira)

Information by:

Name(s): Eng. António Castro

Contact(s):

Appendix B

‘Level 2’ survey form (example)

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CHARACTERISATION OF ADOBE CONSTRUCTION IN AVEIRO DISTRICT

Survey Form **ADOBE BUILDINGS IN THE UNION OF PARISHES OF GLORIA AND VERA CRUZ – ZONE 1**

Note: Each building identified should be marked in colour in the map, with indication of its number (internal code).

No. (internal code)	Building name	Building no.	No. floors	Relative position	In use or vacant?	State of conserv. (1-5)	Year	Is adobe visible?	Is lime or earth mortar visible?	Observations	Photos

Appendix C

‘Level 3’ inspection checklists

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CHARACTERISATION OF ADOBE CONSTRUCTION IN AVEIRO DISTRICT

A Inspection Checklist — GENERAL INFORMATION

1. Inspection

Date: ____ / ____ / ____

	Name	Technical field
Team:	____________________	____________________
	____________________	____________________

2. Location

Parish: ____________________ Municipality: ____________________

Address: ____

3. Contacts

Owner: ____________________ Contact: ____________________

Tenant: ____________________ Contact: ____________________

4. Identification

Building no.: __________

Wall(s) [a] no.: ____________________ ([a] Walls for the delimitation of properties)

Water well(s) no.: ____________________

5. Inspection checklists

Building:

- [X] B01 - Identification of the building
- [ ] B02 - Drawings
- [X] B03 - Identification of the materials of the adobe masonry walls
- [ ] B04 - Evaluation of the roof
- [X] B05 - Evaluation of the facade walls
- [ ] B06 - Evaluation of the floors
- [ ] B07 - Evaluation of the interior walls and ceilings
- [X] B08 - Evaluation of the basement and foundations
- [ ] B09 - Evaluation of other structural elements
- [ ] B10 - Observation of the terrain and vicinity of the building
- [ ] B11 - Interventions in the building
- [ ] B12 - Photographic record

Wall(s):

- [ ] C01 - Identification and evaluation of walls
  Number of C01 checklists filled out: _______
- [ ] C02 - Identification of the materials of the adobe masonry walls
  Number of C02 checklists filled out: _______

Water well(s):

- [ ] D01 - Identification and evaluation of water wells
  Number of D01 checklists filled out: _______
- [ ] D02 - Identification of the materials of the adobe masonry wells
  Number of D02 checklists filled out: _______

6. Additional documentation

Written documentation: ______________________

Graphic documentation: ______________________

Photographic documentation: ______________________

7. References

8. Observations

9. Updates

Date: ____ / ____ / ____

Observations: ______________________

Date: ____ / ____ / ____

Observations: ______________________

Building:	Wall(s):	Water well(s):

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CHARACTERISATION OF ADOBE CONSTRUCTION IN AVEIRO DISTRICT

B01 Inspection Checklist

IDENTIFICATION OF THE BUILDING

1. Building no.: ____________

2. GPS coordinates: ____________

3. Building name: ____________

Photo

4. General information

Year of construction: ________

Author(s): ____________

Type of property:
- [ ] Private property
- [ ] Public property: [ ] Parish [ ] Municipality [ ] State
- [ ] Other

Category:
- [ ] Civil Architecture
- [ ] Industrial Architecture
- [ ] Military Architecture
- [ ] Religious Architecture

Setting:
- [ ] Rural
- [ ] Urban

Architectural typology: ____________

Type of classification:
- [ ] Unclassified property
- [ ] Property about to be classified
- [ ] Property of municipal interest
- [ ] Property of public interest
- [ ] National monument

Type of protection:
- [ ] Not covered by special protection zone
- [ ] Covered by special protection zone

Relative position:
- [ ] Detached
- [ ] Street corner
- [ ] Semi-detached or end-of-terrace
- [ ] Terraced

Building in ruins:
- [ ] Yes
- [ ] No

Number of facades with openings: ________ Which? ____________

5. Use of the building

Building in use:
- [ ] Yes
- [ ] No - **Year of vacancy:** ________

Floor	Celing height	Initial function	Other functions	Current function
		(Commerce / services / single-family residence / multi-family residence / vacant / other)		
Ground	____ m	____	____	____
1^{st}	____ m	____	____	____
2^{nd}	____ m	____	____	____
3^{rd}	____ m	____	____	____
Other:	____ m	____	____	____
Other:	____ m	____	____	____

Ground floor configuration:
- [ ] Partitioning similar to the upper floors
- [ ] With ample interior space (without partitioning)
- [ ] Other: ____________

Possibility of alteration: [ ] Use/funtion [ ] Partitioning

6. Accessibility

Street(s) that allow access to the building:

Name:			Name:		
Width (m): ________	No. of lanes: ______		Width (m): ________	No. of lanes: ______	
Pavement (sidewalk):	☐ Yes	☐ No	Pavement (sidewalk):	☐ Yes	☐ No
Pedestrian crossings:	☐ Yes	☐ No	Pedestrian crossings:	☐ Yes	☐ No

7. Observations

Building:

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CHARACTERISATION OF ADOBE CONSTRUCTION IN AVEIRO DISTRICT

B03 Inspection Checklist — IDENTIFICATION OF THE MATERIALS OF THE ADOBE MASONRY WALLS

1. Adobe units

	Type 1	Type 2	Type 3
Location in the building:	______	______	______
Dimensions:	______	______	______
Colour:	______	______	______
Constituent materials:	______	______	______
	______	______	______
Granulometry:	______	______	______
Mixing ratio:	______	______	______
With identifying mark?	Yes ☐ Which? ______	Yes ☐ Which? ______	Yes ☐ Which? ______
	No ☐ ______	No ☐ ______	No ☐ ______
Source of raw materials:	______	______	______
Manufacturer:	______	______	______

2. Traditional mortars

2.1. Joint mortars

	Type 1	Type 2	Type 3
Location in the building:	______	______	______
Thickness:	______	______	______
Colour:	______	______	______
Constituent materials:	______	______	______
	______	______	______
Granulometry:	______	______	______
Mixing ratio:	______	______	______
Source of raw materials:	______	______	______

2.2. Render and plaster mortars

2.2.1. Render (exterior)

	Type 1	Type 2	Type 3
Location in the building:	______	______	______
Thickness:	______	______	______
Colour:	______	______	______
Constituent materials:	______	______	______
	______	______	______
Granulometry:	______	______	______
Mixing ratio:	______	______	______
Source of raw materials:	______	______	______

2.2.2. Plaster (interior)

	Type 1	Type 2	Type 3
Location in the building:	______	______	______
Thickness:	______	______	______
Colour:	______	______	______
Constituent materials:	______	______	______
	______	______	______
Granulometry:	______	______	______
Mixing ratio:	______	______	______
Source of raw materials:	______	______	______

3. Observations
Building:

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CHARACTERISATION OF ADOBE CONSTRUCTION IN AVEIRO DISTRICT

B05 Inspection Checklist

EVALUATION OF THE FACADE WALLS

1. Structural system

1.1. Type:

Adobe masonry ☐ thick.: _____cm | Stretcher bond ☐ | English bond ☐ | Header bond ☐ | Other: __________ ☐

↓ Facade(s): ____________________

Stone masonry ☐ thick.: _____cm | Mortared ☐ | Rubble masonry ☐ | Ashlar masonry ☐ | 'Eirol' stone ☐ | Schist ☐ | Other: __________ ☐ __________

↓ Facade(s): ____________________

Ceramic brick masonry ☐ thick.: _____cm | Single-leaf ☐ | Double-leaf ☐ | Hollow bricks ☐ | Solid bricks ☐

↓ Facade(s): ____________________

Reinforced concrete ☐ thick.: _____cm → Facade(s): ____________________

Other: ______________ ☐ thick.: _____cm → Facade(s): ____________________

1.2. Connection between facade walls: ____________________

____

1.3. Structure of the facade wall openings: ____________________

____

1.4. Existence of heavy equipment connected to the building envelope: Yes ☐ No ☐

1.5. Function:

	Facade walls			
	Main	Rear	Left	Right
Load-bearing				
Non-load-bearing				
Uniform thickness along the height				
1.6. State of conservation (1 - 5):				

(1: very poor; 3: reasonable; 5: very good)

2. Exterior wall finishes

2.1. Type:

	Facade walls			
	Main	Rear	Left	Right
Textured paint				
Water-based 'plastic' paint				
Lime paint				
Ceramic tiles: Unglazed				
Ceramic tiles: Glazed				
Lime mortar				
Earth mortar				
Cement mortar				
Stone veneers				
Special/decorative elements (______________)				
Other: ______________				
2.2. State of conservation (1 - 5):				

(1: very poor; 3: reasonable; 5: very good)

3. Defects

3.1. Cracking

3.1.1. Schematic representation of the cracking observed:

Main facade	Rear facade
Left side facade	Right side facade

		Facade walls			
		Main	Rear	Left	Right
3.1.2. Pattern of cracking along the height:	Concentration at the base				
	Concentration at the top				
	Uniformly distributed				
3.1.3. Pattern of cracking along the length:	Concentration at the left side				
	Concentration at the middle				
	Concentration at the right side				
	Uniformly distributed				
3.1.4. Orientation of cracks:	Approximately horizontal				
	Approximately vertical				
	Diagonal				
3.1.5. Concentration of cracks near openings:					
3.1.6. Orientation of cracks near openings:	Approximately vertical				
	Approximately horizontal				
	Diagonal				
3.1.7. Possible causes of cracking:	Excessive load imposed by the roof or floor structures				
	Horizontal thrust imposed by the roof structure				
	Deformations due to variations in temperature or moisture content				
	Localised cracking, due to stress concentration (Where? ______)				
	Differential foundation settlement				
	Incompatibility between the behaviour of different structures or materials				
	Drying shrinkage of mortar				
	Reaction to salts (efflorescence/cryptoflorescence)				
	Other: ______				
3.2. Leaning of facade walls:					
3.3. Horizontal displacement of facade walls:	Increasing along the height				
	Predominately at the level of a floor (Which floor? ______)				
	Other situation (create a schematic representation below)				

		Facade walls			
		Main	Rear	Left	Right
3.4 Bulging of facade walls (create a schematic representation below):					
Due to:	Excessive load				
	Soil expansion				
	Horizontal thrust				
3.5 Differential settlements:					
3.6 Defects in corners:	Cracking between walls				
	Separation between walls				
	Other:__________________				
3.7 Loss of squareness in openings:					
3.8 Excessive bending of the lintels:					
3.9 Dampness:	Rising damp:				
	Visible on: Outer surface				
	Visible on: Inner surface				
	Dampness at the top of walls:				
	Visible on: Outer surface				
	Visible on: Inner surface				
	Roof parapet acts as a barrier to water drainage				
	Dampness near the roof of adjacent lower height building section				
	Dampness in the wooden lintels				
	Water penetration through window openings				
	Surface condensation				
	Other:__________________				
3.10 Other:	Render detachment				
	Render erosion				
	Paint peeling				
	Paint erosion				
	Ceramic tile deterioration				
	Aging of materials				
	Polution, graffiti, moss, mould				
	Other:__________________				

3.11 Schematic representation of the defects observed (when necessary):

4. Observations:

Building:

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CHARACTERISATION OF ADOBE CONSTRUCTION IN AVEIRO DISTRICT

B08 Inspection Checklist

EVALUATION OF THE BASEMENT AND FOUNDATIONS

1. General information

Number of underground floors: ☐

Coincidence, in plan view, of the basement with the rest of the building: ☐ Yes
☐ No → Schematic representation:

Signs of excavations subsequent to construction: ☐ No
☐ Yes → Description: ____________________
____
____
(Locate them in the previous scheme)

Signs of changes in the level of the water table: ☐ No
☐ Yes → Description: ____________________
____

2. Foundations

2.1. Type of foundation:
☐ Not possible to identify
☐ Adobe masonry
☐ Stone masonry - Stone: ____________
☐ Reinforced concrete footing
☐ Other: ____________

2.2. State of conservation (1 - 5): ☐ (1: very poor; 3: reasonable; 5: very good)

2.3. Foundation defects:
☐ Lowering of the ground floor level
☐ Vertical displacement of the terrain
☐ Horizontal displacement of the terrain
☐ Differential settlements
☐ Other: ____________

Schematic representation of the defects (if necessary):

3. Walls

3.1. Existence of earth retaining walls: ☐ Yes ☐ No

3.2. Type:
- ☐ Not possible to identify
- ☐ Adobe masonry
- ☐ Stone masonry
- ☐ Ceramic brick masonry
- ☐ Reinforced concrete
- ☐ Other: ____________
- ☐ Coating: Material: ____________ Thickness: ____________
- ☐ Joints: Material: ____________ Thickness: ____________

Buttresses: ☐ Yes ☐ No

3.3. Dimensions/inclination:

Thickness: Base ☐ cm Top ☐ cm

Height: _______ m

Inclination: ☐ No ☐ Yes - _______ º

3.4. State of conservation (1 - 5): ☐ (1: very poor; 3: reasonable; 5: very good)

3.5. Walls defects:
- ☐ Damp patches
- ☐ Cracking
- ☐ Disintegration of the material
- ☐ Crushing
- ☐ Other: ____________

Schematic representation of the defects (if necessary):

3.6. Consolidation/shoring of the walls: ☐ No ☐ Yes → Schematic representation:

4. Observations

Building: